DERWENT

Design for a Steel Dam

Civil Engineering

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DESIGN

FOR A

STEEL DAM

BY

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This is to certify that the thesis prepared under the immediate direction of Assistant Professor F. O. Dufour by

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entitled DESIGN OF A STEEL DAM

is approved by me as fulfilling this part of the requirements for the Degree of Bachelor of Science in Civil Engineering.

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INTRODUCTION

Geo. S. Morison in his book The New Epoch says: "Man's capacity is no longer so limited; he has learned to manufacture power, and with the manufacture of power a new epoch began." This is an age in which mankind is turning eagerly to the utilization of all the great potential forces and resources of nature. Everywhere companies are being organized, and schemes promoted for the manufacture of power as a commercial commodity. The perfection of electric transmission has made it practicable to install stations for the generation of electric power at some distance from the place where it is to be used, and has thus greatly stimulated the development of water power.

The soil of the great arid regions in parts of our Western States contains all the elements necessary except moisture to make it abundantly productive. To supply this moisture, much engineering work is now in progress and much more will be commenced in the next few years.

Almost every instance of these two great branches

of civic improvement, water power development and irrigation, requires the construction of some kind of a dam, either to control a natural waterfall or to create an impounding reservoir. The water supply systems for a great many cities also call for the construction of dams. It is thus seen that the design and construction of dams, as a part of the civil engineer's work, is becoming very important.

Nearly all of the common structural materials are used in dams; but it is the purpose of this article to describe to some extent the use of steel as a material for dams, and to make a design and estimate for a steel dam for a site now occupied by one of masonry.

STEEL VERSUS MASONRY

There are two prime requisites for any dam. It must be more or less impervious, and stable enough to resist the pressure and eroding action of the water. Steel has the advantage of being absolutely impervious under all pressures; and it is not damaged by the action of water flowing over it or through a leak. The pressures in a steel dam may be calculat-

ed exactly; and, since the strength of the material is known, all stresses may be definitely provided for. On the other hand, nearly all masonry dams are not absolutely impervious, and their stability is dependent to a great extent upon whether or not water is allowed to get into or under the masonry; and besides, neither the disposition of the stresses or the behavior of the material are definitely known.

The steel dam may be erected in much less time than is required for one of any other material, which is a great advantage in a case where the time for construction is limited, or the locality unhealthy. The erection of the steel work does not call for a large plant, such as used in making and placing concrete or in laying large stones. Unless the site is very far removed from steel mills and the stone is near by and easily quarried, the steel dam is cheaper. The steel should last as long as masonry, if properly protected and occasionally painted. It is not claimed that a steel dam is always, or even usually superior to one of masonry or concrete, but for some conditions and uses the steel has advantages over other materials which at

least demand consideration.

THE USE OF STEEL FOR DAMS

Structural steel seems not to have been used as a material for dams until comparatively recently, and its use is still almost wholly confined as an auxiliary method in rendering earth or rock dams impervious, to the reinforcing of concrete dams, or to the various forms of movable dams used in controlling works.

The Lower Otay dam at Otay, Cal., is a good example of the use of steel to render impervious an otherwise pervious type of dam. This structure is described by W. S. Russel in the Engineering News, Mar. 10, 1898. It is a loose rock dam, 130 feet high and 545 feet long, with a core wall of steel plates from No. 0 to No. 3 Birmingham Gage riveted and calked together with as much care as is taken in constructing a boiler for high pressure. The steel is placed in a vertical position and protected by asphaltum and a thin wall of concrete.

A similar dam was built at East Canyon Creek, Utah, in 1900. It was later enlarged and

the steel core wall extended in the form of an inclined steel face on the water side.

An example of steel facing on a loose rock dam is seen in the Goose Neck Canyon dam. This dam is 210 feet high, faced with 3/8 inch steel plates on a slope of 2 to 1.

In 1897, a peculiar design was considered for a dam for the Pioneer Electric Power Co. at Ogden Utah. The dam was to be 400 feet long and 60 feet high. Geo. H Pegram and Henry Goldmark proposed to build a series of concrete piers $11\frac{1}{2}$ feet thick supporting arched steel plates from $\frac{1}{2}$ to $\frac{7}{16}$ inches thick with a clear span of 23 feet.

Many other examples of the use of steel in fixed dams, and many more of its use in movable dams might be cited; but reference will be made to only two more. The first fixed metallic dam to be built was erected by the Atchison, Topeka and Santa Fe. Ry., four miles east of Ash Fork, Arizona. It consists of 24 right-triangular steel bents resting on concrete foundations and supporting $\frac{3}{8}$-inch curved steel plates on the upstream sides of the

bents. These bents are spaced 8 feet apart, and the water face is inclined at an angle of 45°. The dam was designed by F.H. Bainbridge and is described in detail in the Engineering News for May 12, 1898. F.H. Bainbridge also has a discussion of steel dams in the Engineering News for Sept. 28, 1905.

A steel dam similar to the one at Ash Fork was built in 1901 at Redridge, Mich. It differs from the Ash Fork dam in being anchored to a heavy concrete base instead of to the bed rock. The whole section thus acts as a gravity dam. The Redridge dam was designed by J. F. Jackson, Assoc. M. Am. Soc. C.E.

As far as the writer has been able to determine, these two are the only fixed structural steel dams in existence.

DESIGN OF A STEEL DAM

LOCATION OF THE DAM

For the purpose of making a comparison between a steel and a masonry dam, it was decided to select a completed masonry dam and design a steel structure for the same site. The masonry dam chosen is located on the Middle Fork of Crow Creek, 12 miles from Cheyenne, Wyoming. The reservoir thus formed has a storage capacity of about 1700 million gallons, and is used to supplement the water supply of the city of Cheyenne.

THE MASONRY DAM

This dam was built in 1903 and 1904 by Gaffy & Keefe, for the city. It is situated at the narrowest place in the canyon, where the creek flows over bare granitic rock, and the slope rises at the rate of two and one fourth horizontal to one vertical. The dam is constructed throughout of uncoursed rubble masonry laid in portland cement mortar. In plan the dam is curved, the radius being 300 feet. It's extreme height from foundation to par-

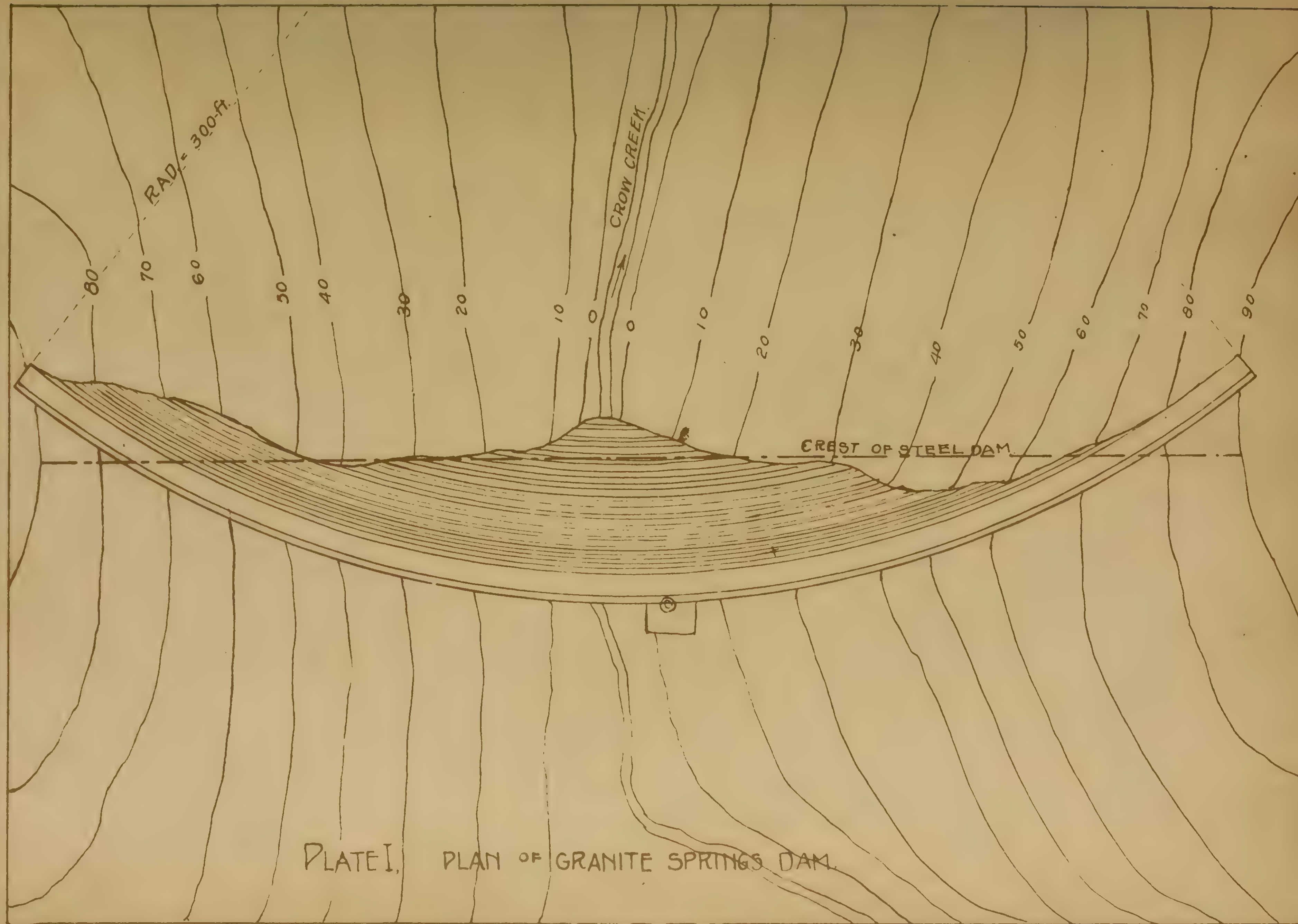


PLATE I, PLAN OF GRANITE SPRINGS DAM.

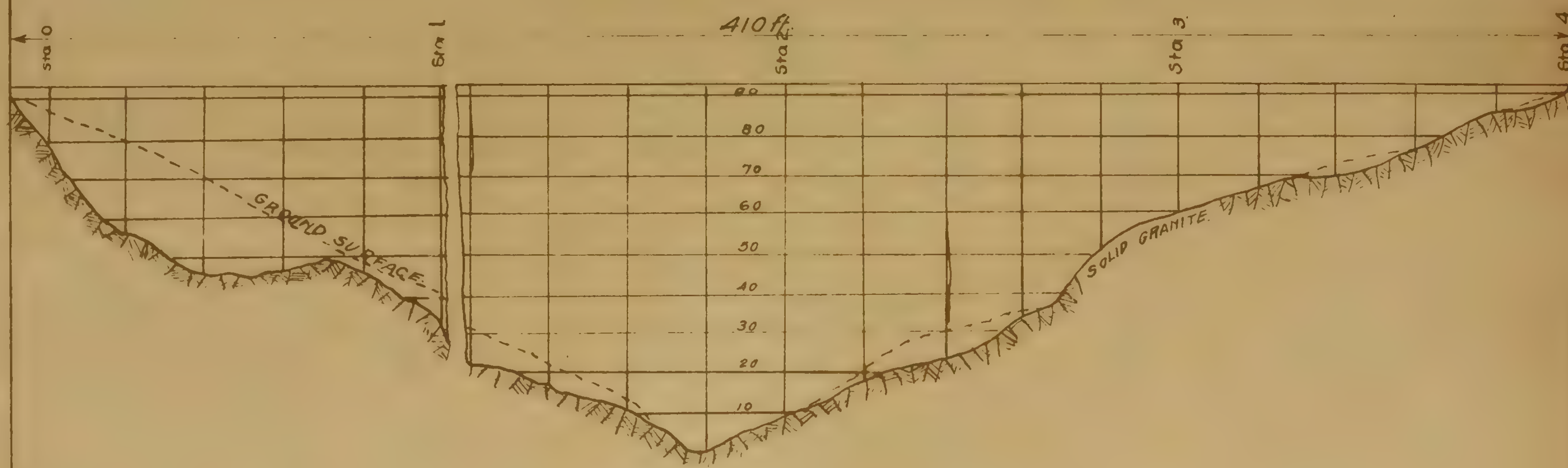


PLATE II ELEVATION OF GRANITE SPRINGS DAM.

apex is 96 feet, the length at the base 10 feet, and the length along the crest 410 feet. The structure contains 14 477 cubic yards of masonry.

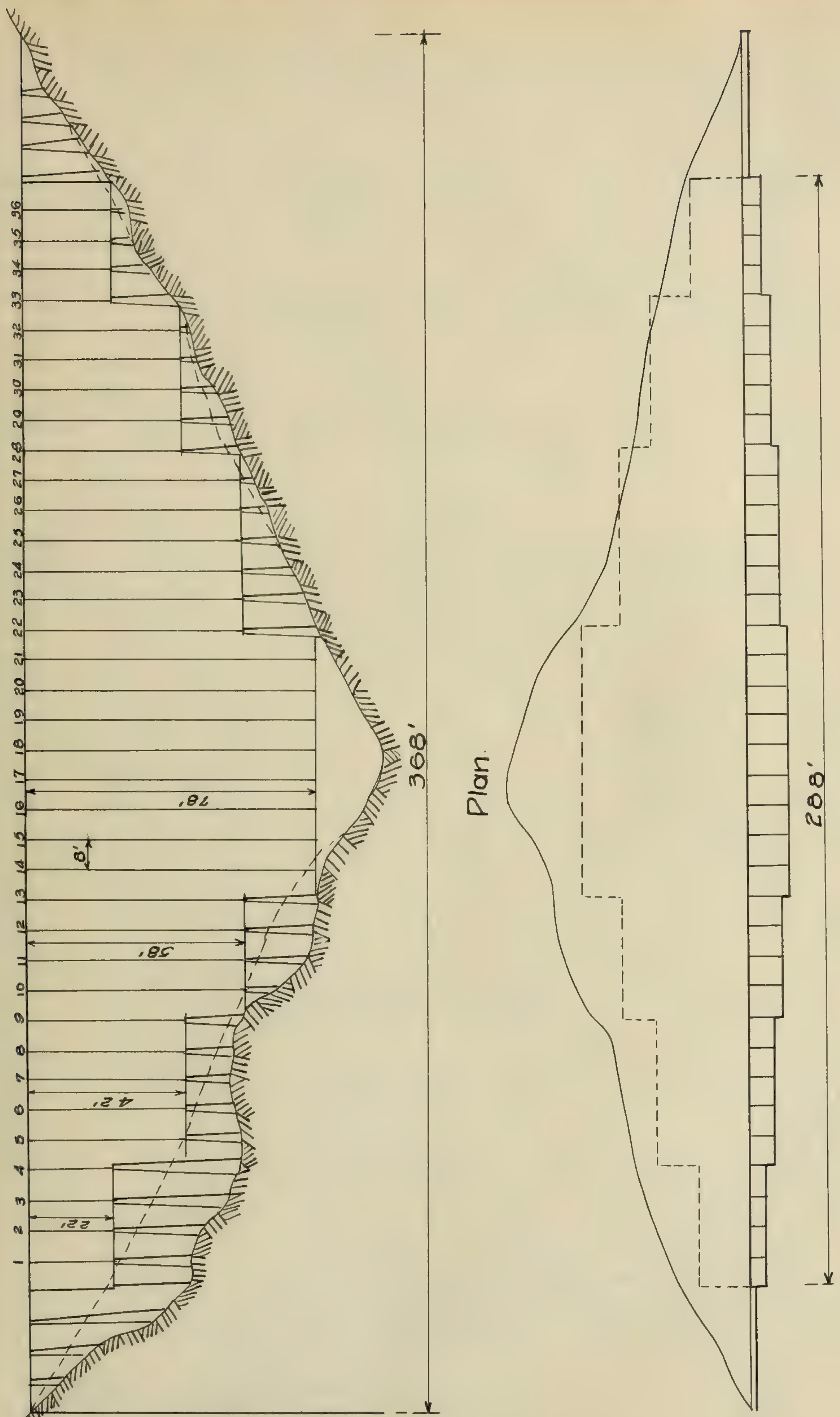
Plates 1 and 2 show the plan and elevation of the dam.

FOUNDATION ROCK.

The steel dam will be designed for the same site, as nearly as possible, as that occupied by the present dam. The rock on which most of the dam rests is a dense granite, entirely free from seams and cracks, and "vitreous in its hardness". On the east end, above elevation 50, the rock is disintegrated, and requires considerable excavation to obtain suitable foundation; at the west end, however, the hard rock is exposed at the surface nearly to the top of the dam.

GENERAL LAYOUT OF SUPERSTRUCTURE

The steel-work of the dam consists of thirty-six A-shaped frames or bents, of four different sizes, ranging in height from 22 feet to 78 feet, and supporting on their upstream legs a steel apron made up of curved plates riveted to the bents. The position of the


Down-Stream Side.
368'
78'
58'
42'
22'
8'
Plan.
288'

bents is illustrated in the sketch on page 9. Each bent is made up of standard rolled forms. The upstream leg is inclined at an angle of 30° and the downstream leg at an angle of 8° with the vertical. These bents are spaced 8 feet apart and connected in pairs by sway bracing.

Riveted between the bents are $\frac{3}{8}$-inch steel face-plates, concave on the water side, and 7 feet $4\frac{1}{2}$ inches across the chord. They are curved with an 8-foot radius. These plates are riveted to splice plates 18 inches wide and ½ inch thick which cover the I-beams, (and are riveted to their flanges), which form the front legs of the bents. All joints where plates are spliced are calked so as to be water tight.

THE SUBSTRUCTURE

The rock on which the dam stands is firm enough to serve as foundation and anchorage for the steel without interposing any masonry between, but in order to reduce the number of sizes of bents, and thereby the cost of both shop work and drafting, it is decided to make but

four sizes of frames and support them upon concrete piers. A good quality of portland cement concrete is used. The unsound rock is excavated and the upstream face of the pier is anchored to solid rock. Where the exposed rock has been worn smooth by the action of the elements, it is roughened by pointing or shallow shooting before any concrete is laid. Concrete abutments are to be built out at each end of the dam for about 30 feet. The water face of all concrete work has the same slope as the steel (30° with the vertical); and is covered with a ¼ inch steel apron. The latter is riveted to Z-bars embedded in the masonry, the joints are calked water tight, and it extends into and makes a permanently water tight joint with the rock at the bottom and sides of the canyon. The concrete work will now be discussed more in detail.

THE ABUTMENTS

The abutments for the two ends are much alike both in length and height. The maximum height above the ground surface is in

each case about 23 feet. The upstream face has a 30° slope. The top of the section is 18 inches wide and the back face is vertical except for buttresses placed at intervals of 8 feet. These buttresses have a slope on the downstream side of 1 to 4 as shown in Figs. 1 and 2. As a precaution against sliding, a trench about 1 foot deep and 3 feet wide is excavated in the rock under the downstream side of the main part of the wall. The top of the abutment is finished with a coping which is 12 inches thick and projects 3 inches from the face of the wall on both sides.

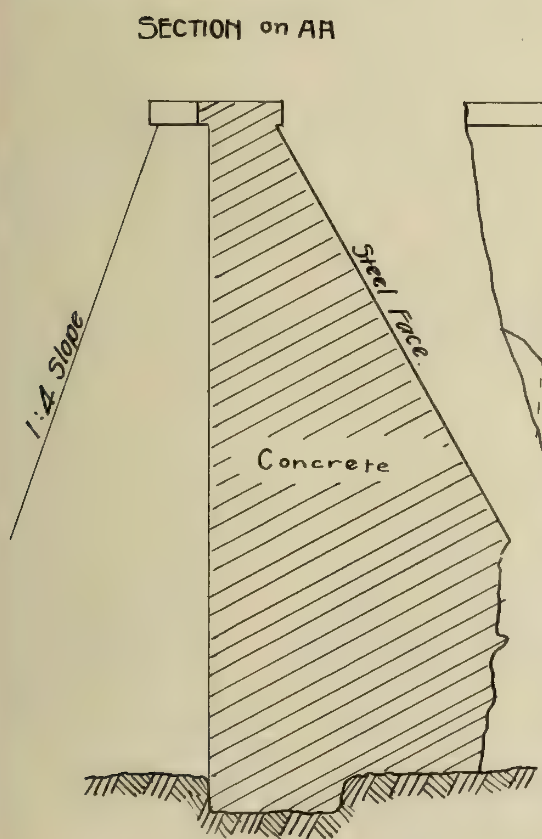


Fig. 1

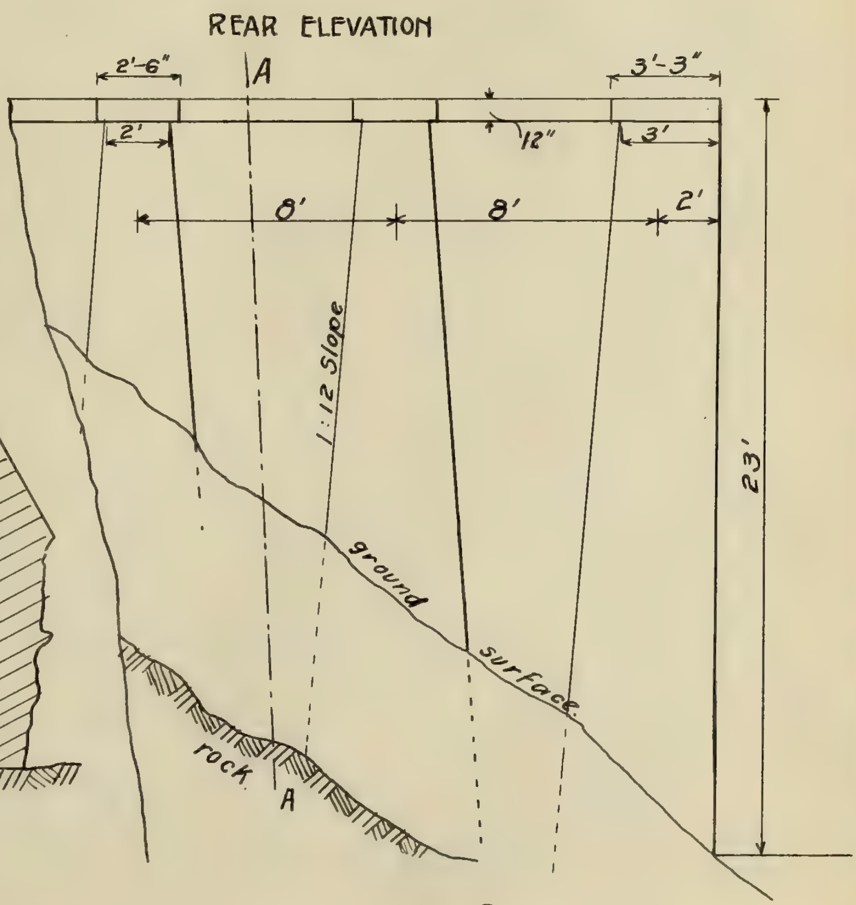


Fig. 2

PIERS FOR 22-FT. BENT

The shortest bents, those next the abutments, and designated on page 9 as 1, 2, 3, 4, 33, 34, 35 and 36, are all alike; and each is 22 feet high. The spread of the frame at base $(22 \cdot \tan 30° + 22 \cdot \tan 8°) =$ 15 feet, 10 inches. The pressure due to weight of steel and water is concentrated at the back of the pier where two columns are supported as shown in Fig. 3. . . The sum of the stresses in these two columns = 111,000 lb. The allowable bearing pressure on masonry is taken at 250 lb. per sq. in. $111,000 \div 250 =$ 445 sq. in. required. $\sqrt{445} = 21$. The bearing surface for these two columns must be about 21 inches square. The downstream end of the pier will be made 32 inches square on top, and its upper surface will be inclined at an angle of 30° with the horizontal.

The piers are connected at their upstream ends by a concrete wall, 5 feet wide on top, the water face having the prevailing slope of 30° with the vertical, and the downstream face a 2 : 12 slope. From 1 to 2½ feet of the I-beam which forms the upstream leg of the bent, is embedded in the

face of the wall, and anchor bars run from the end of the beam down into bed rock.

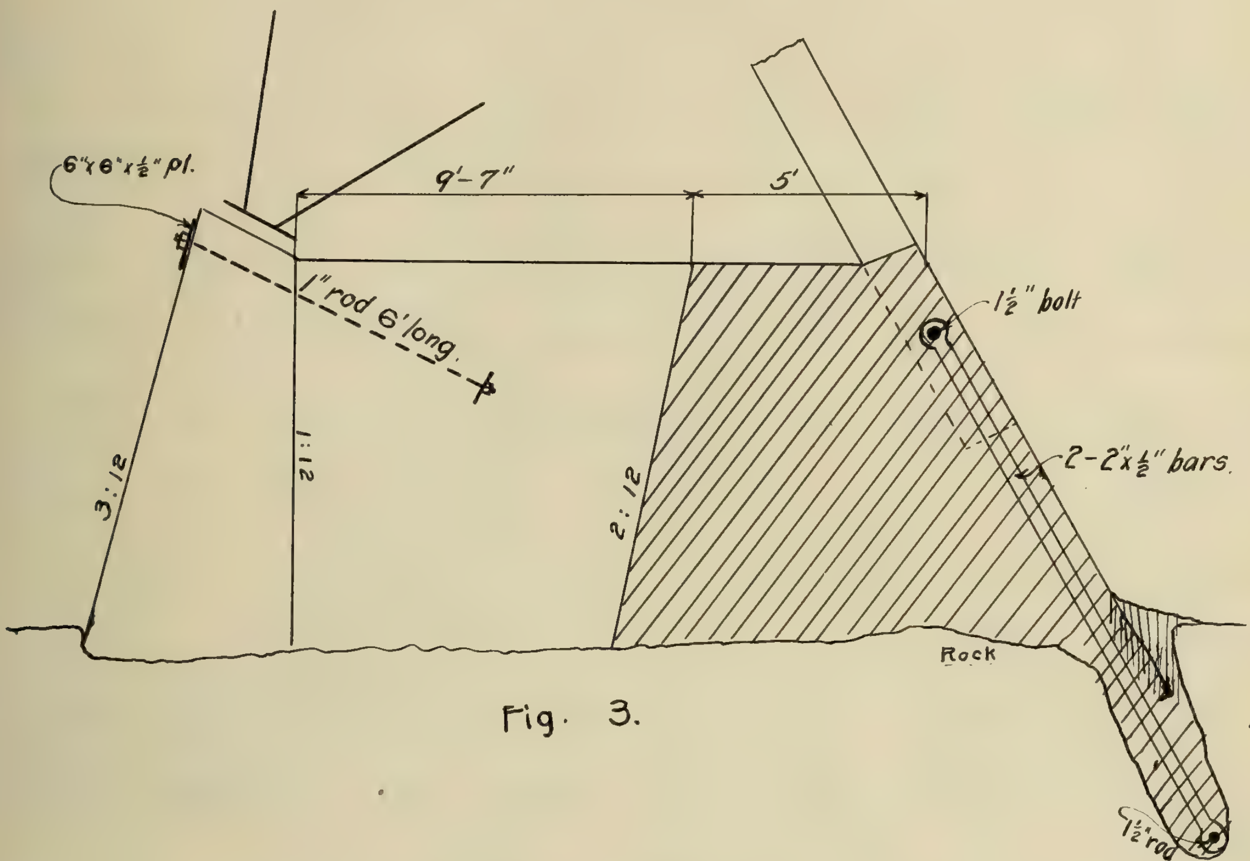


Fig. 3.

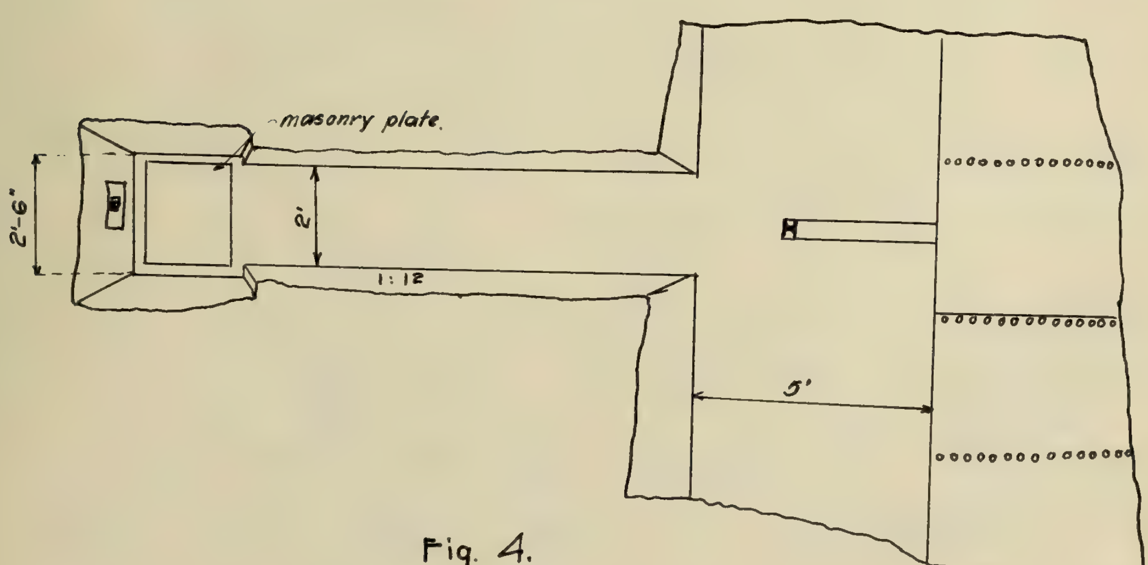


Fig. 4.

Fig. 3 is an elevation of one pier with section of the connecting wall; Fig. 4. is a plan of the same.

PIER FOR 42-FT. BENT

The next size of bent is 42 feet high. Bents Nos. 5, 6, 7, 8, 9, 28, 29, 30, 31, and 32 are this size. The spread of frame at the base = 32 feet, 2 inches. The sum of the pressures transmitted by the two rear columns to the masonry is 243,000 lb., (see page 38 for computation of the stresses). 243,000 ÷ 250 = 970 sq. in. = the required bearing area. A bearing 27 x 36 in., giving an area of 972 sq. in is used. The pressure transmitted to the middle of the pier by the third column is 217,000 lb. The required bearing area = 217,000 ÷ 250 = 870 sq. in. 27" x 33" = 890 sq. in., and this size of bearing will be used.

The end pier will be 36 x 42 inches at the top, with its top face inclined at 30° with the horizontal; and the middle pier will be 36 x 40 inches on top with its surface inclined at an angle of 35° with the horizontal. The walls connecting the piers will be 2 ft. wide on top and the wall at the face of the dam will be

7 ft. wide on top, all slopes being the same as in the substructure for the 22-ft bent

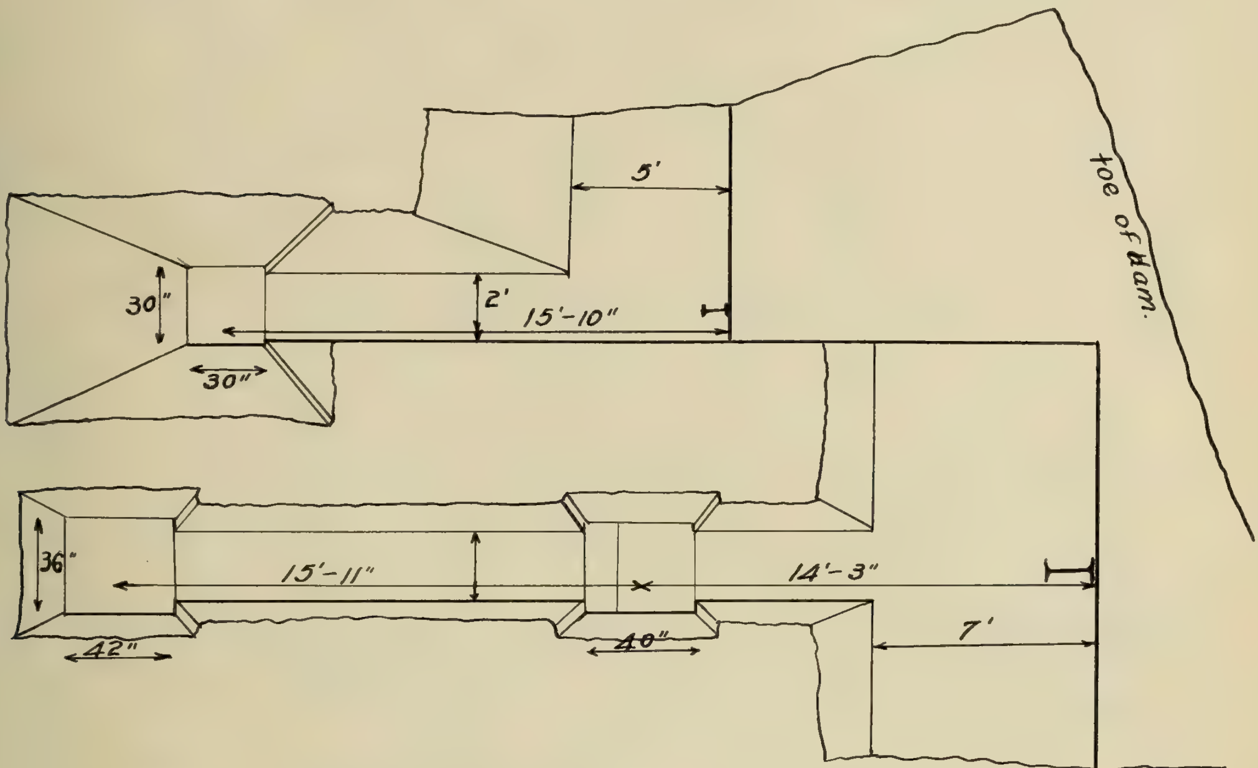


Fig. 5.

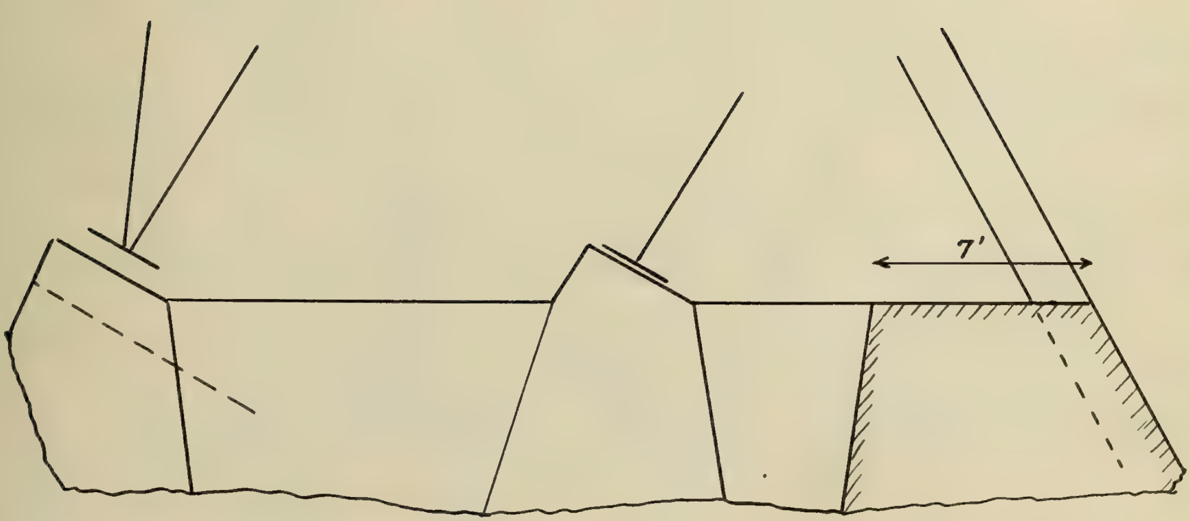


Fig. 6.

PIER FOR 58 FT. BENT

Bents No. 10, 11, 12, 13, 22, 23, 24. 25, 26 and 27 are all 58 ft. high. The totat spread at the base of each frame is 41 ft., 7 inches. The pressure transmitted to the masonry by the two rear columns is 487,000 lb., (see page 48 for computation). 487,000 ÷ 250 = 1,950 sq. in required in the bearing. A bearing 45 x 45 in. giving an area of 2,030 sq. in. will be used. The other two columns give pressures of 220,000 lb. and 233,000 lb., respectively. For the first, 220000 ÷ 250 = 880 sq. in., and a 27 x 33 in. pedestal will be used. For the second, 233,000 ÷ 250 = 930 sq. in., and a pedestal 27 x 36 in. is required.

The top of the end pier will be 50 x 58 in. on a 30° slope. The top of each of the two middle piers is to be 40 x 36 in., the first making an angle with the horizontal of 54°, and the second 60°, (see Fig. 7 for dimensions etc.). The side slopes are the same as in the foundations for the 22-ft. and the 42-ft. bents.

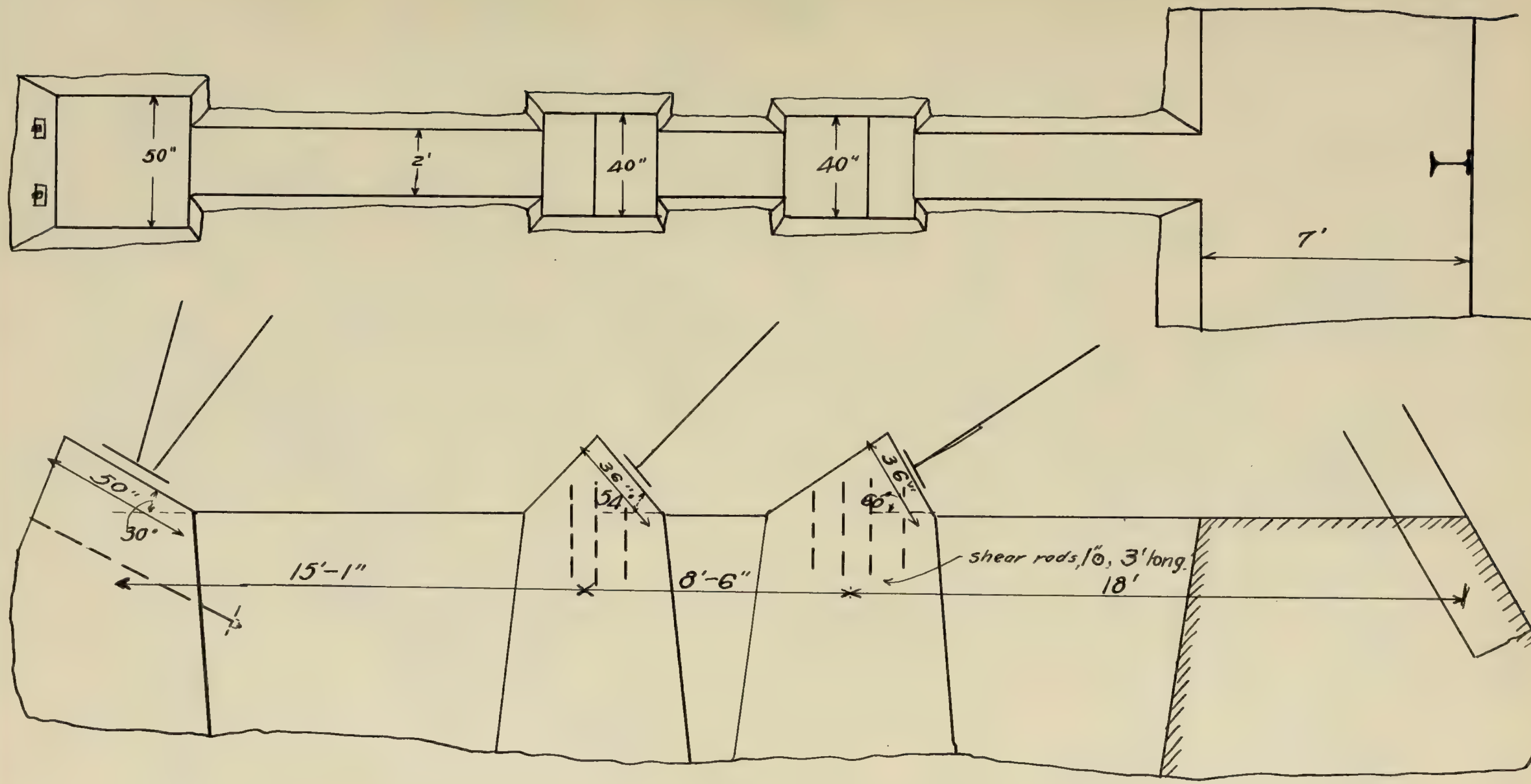


Fig. 7.

FOUNDATION FOR 78-FT. BENT

Under the highest bents no piers are built but the whole space between the base of the steel-work and the bed rock is filled in solidly with concrete. The width of a 78-ft. frame at the base is 56 ft. Three columns meet at the downstream side of the base; and three others are supported between these and the water face, as shown in Fig. 8. The sum of the pressures carried to the masonry by the three rear columns is 957,000 lb., (see page 61); and this divided by 250 = 3,820 sq. in. which is the required bearing. A pedestal 72 x 54 in. giving an area of 3,900 sq. in. will be used. The other columns carry from 248,000 lb. to 263,000 lb. and 263,000 ÷ 250 = 1,050 sq. in. is the required bearing area. A pedestal 30 in x 35 in gives 1,050 sq. in., and this size is used for each of the three columns.

Inclined bearing surfaces are built in the concrete to support the columns. The dimensions of these are shown in Fig. 8.

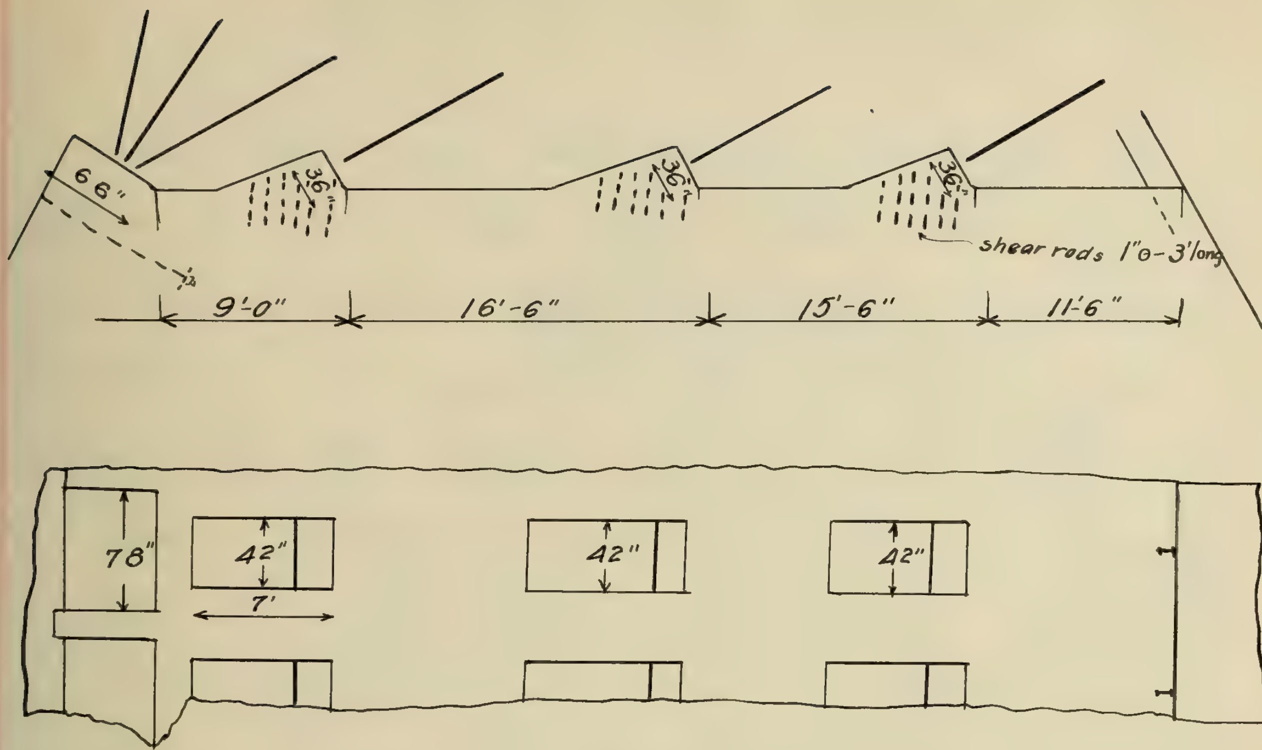


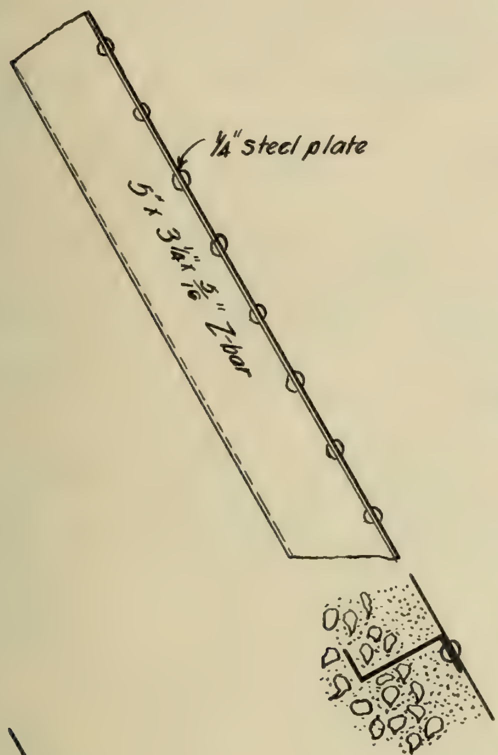
Fig. 8.

Taking the shearing strength of concrete at 50 lb. per sq. in., it is found that some reinforcement is necessary to prevent the raised portions from shearing off under the pressure from the columns. This is taken care of by 12 rods of 1 sq. in. cross-section placed in a vertical position in the concrete, as shown in the figures.

Steel Facing on Concrete

The ¼-inch steel plate on the water face of the wall is in sheets 6 feet wide. These sheets are riveted to 5"x 3¼"x $\frac{5}{16}$" Z-bars, spaced 3 feet apart vertically and embedded in the concrete. These plates are placed in position and the bottom ones riveted on before the first forms are put in, and then the remaining plates are put on as the wall is built up. Two inches of 3 to 1 portland-cement mortar is laid next to the steel, the concrete being tamped up closely behind the plates so as to leave no air spaces. ⅝-inch rivets are used for this steel work. They are spaced 5 inches apart except at joints. The plates are connected to each other by single riveted lap-joints along the vertical edges, and by double riveted butt joints along the horizontal edges. All of the joints are calked water tight. Fig. 9 shows how the plates are connected to the Z-bars and to each other. At the bottom and the sides of the canyon the steel plate is let into the rock by cutting a channel 6 inches wide and 18 inches deep, and embedding the edge of the plate in this with asphaltum concrete. A 3½"x 3"x $\frac{5}{16}$" L is riveted to

the edge of the plate to assist in holding it to the concrete.



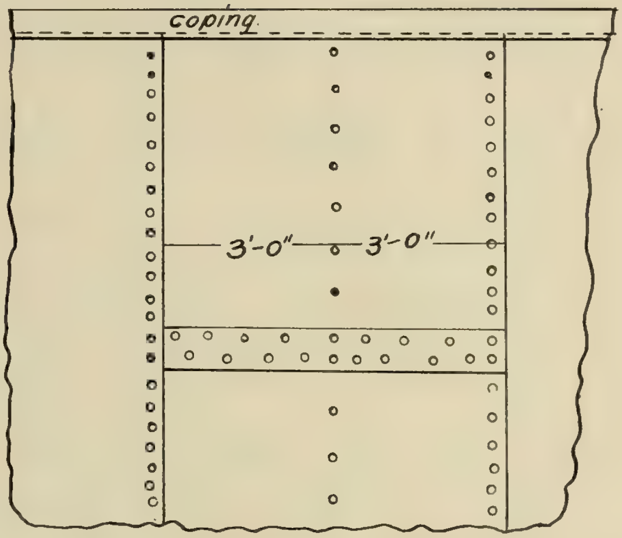


Fig. 9.

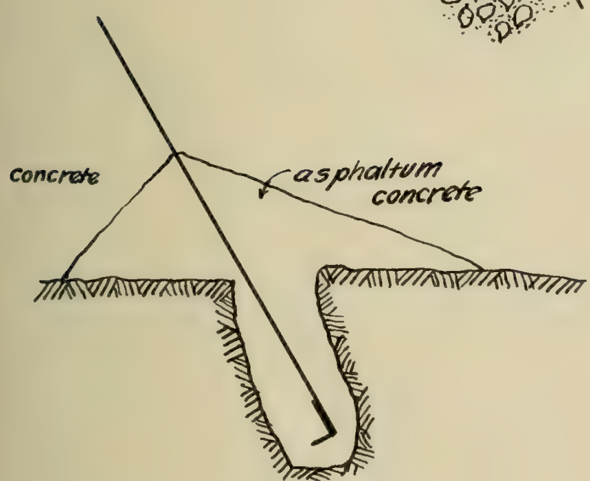


Fig. 10.

Fig 10 illustrates the union of the steel face-plate with bed rock.

The end abutments must be connected rigidly to the steel plates: and the connection must not be such as to cause undue strains or cracks in the concrete. This union is effected by riveting the first curved plate to a 6"x 3½"x 3/8" Z-bar, the latter

being embedded in the face of the abutment, 5 inches from the end and being also anchored firmly to the concrete of the abutment. Slots 1"x3" are cut in the web of the Z-bars, 3 inches from the outer flange and 4 feet apart, through which loops of 3/4-inch rods are passed and secured by pins of the same material. The other end of the rod is threaded and carries a nut and washer. Between each pair of these rods, holes 5 inches in diameter are cut in the web of the Z-bar, and through these the concrete is bonded.

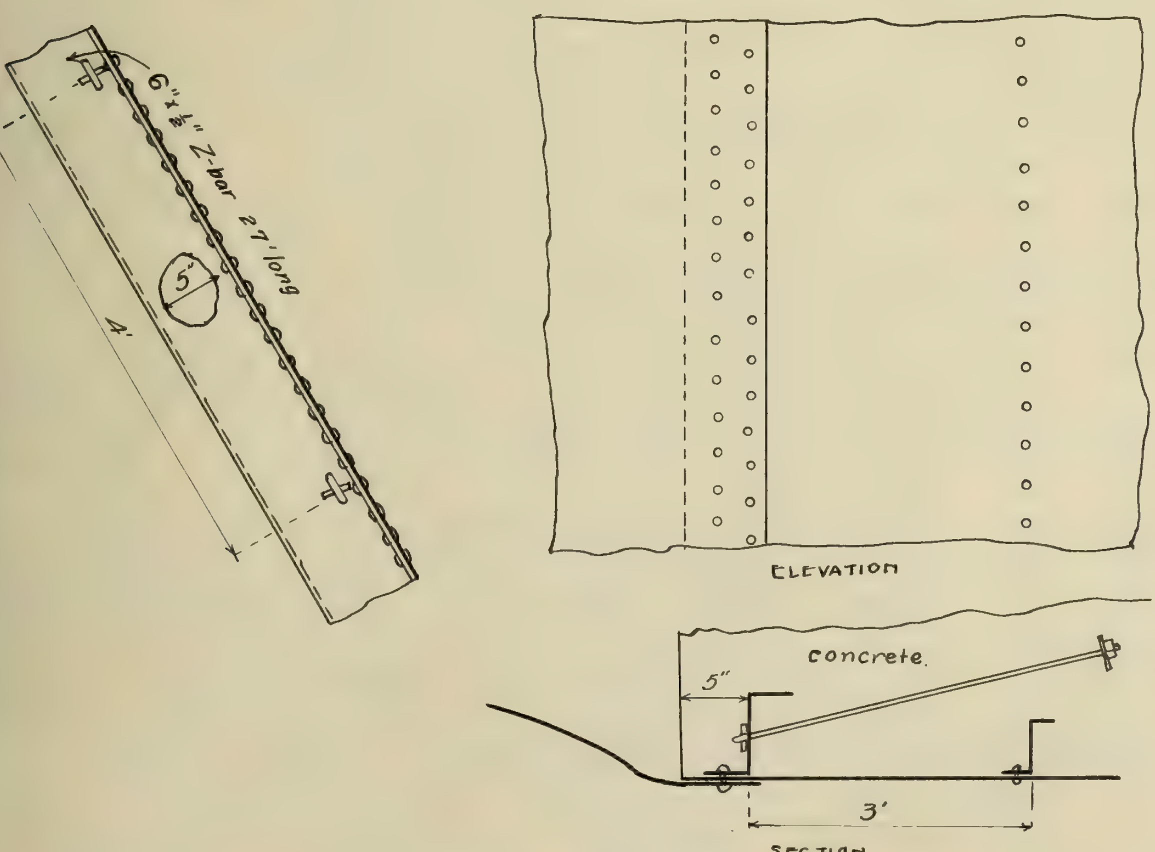


Fig. 11

STEEL STRUCTURE

SPECIFICATIONS.

Cooper's 1900 Specifications for Highway Bridges are used. Medium open hearth steel will be assumed in the design. Dimensions, areas and radii of gyration of rolled sections are taken from the Carnegie Pocket Companion. All loads will be considered as dead loads.

THE 22-FT. BENT.

The general layout of the steel bents has been described on page 8 and illustrated on page 9. The first frames or bents from the ends are 22 feet high. There are eight of these,— numbered on the sketch, 1, 2, 3, 4, 33, 34, 35, and 36. All these bents are spaced 8 feet apart, center to center. The front, or water side of the bent makes an angle of 30° with the vertical, and the downstream leg makes an angle of 8° with the vertical. The weight of water is taken at 62.5 lb. per cu. ft.

Fig. 12 is a stress sheet of the 22-ft bent. The length of the upstream leg is $22 \div \cos 30° = 25.4$ feet =

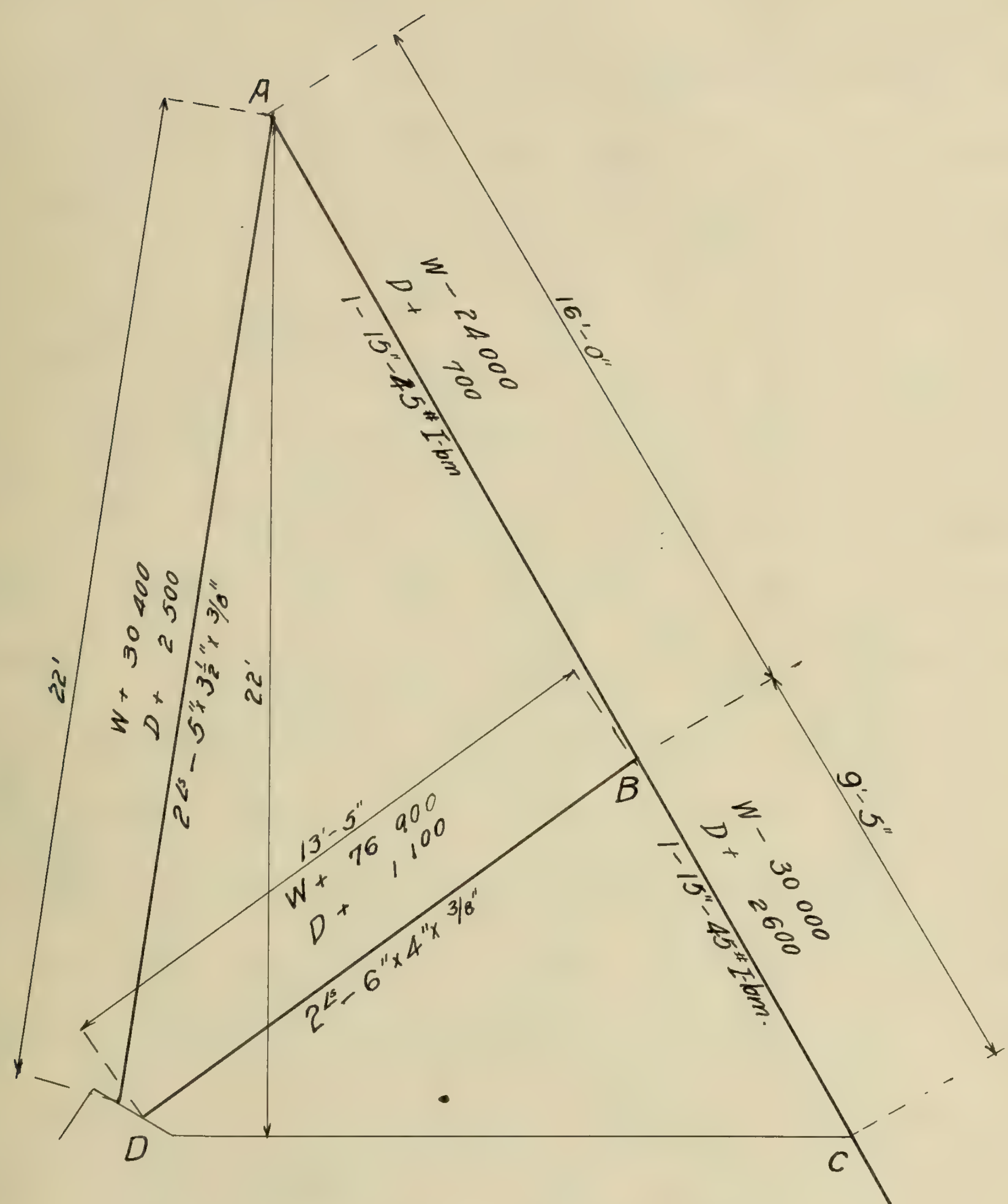


Fig. 12. Stress Sheet for 22-ft Bents.

25 feet, 5 inches. The length of the downstream leg is $22 \div \cos 8° = 22.2$ feet $= 22$ feet, $2\frac{1}{2}$ inches. The upstream leg is constructed of I-beams supporting steel plates. The water pressure in lb. per sq. ft. at the bottom of the bent is $22 \times 62.5 = 1{,}380$ lb. per sq. ft. The upstream leg is divided into two panels. In order that the bending moments in adjacent panels may be approximately equal, and thus make possible the use of a beam of the same section for both panels, the total length of face is divided unequally as shown. Several trials were usually necessary to determine these lengths. Only the last will be given here. The water pressure at the bottom of the bent is laid off graphically as is shown in Fig. 13, and values for pressure at all other points desired is scaled from the figure. The shaded areas on the figure represent graphically the total water pressure on the two panels. The maximum bending moment in each beam is opposite the center of pressure which is at the center of gravity of the corresponding area. The area above the center of pressure in each panel is also shown on the diagram, with its respective center of gravity.

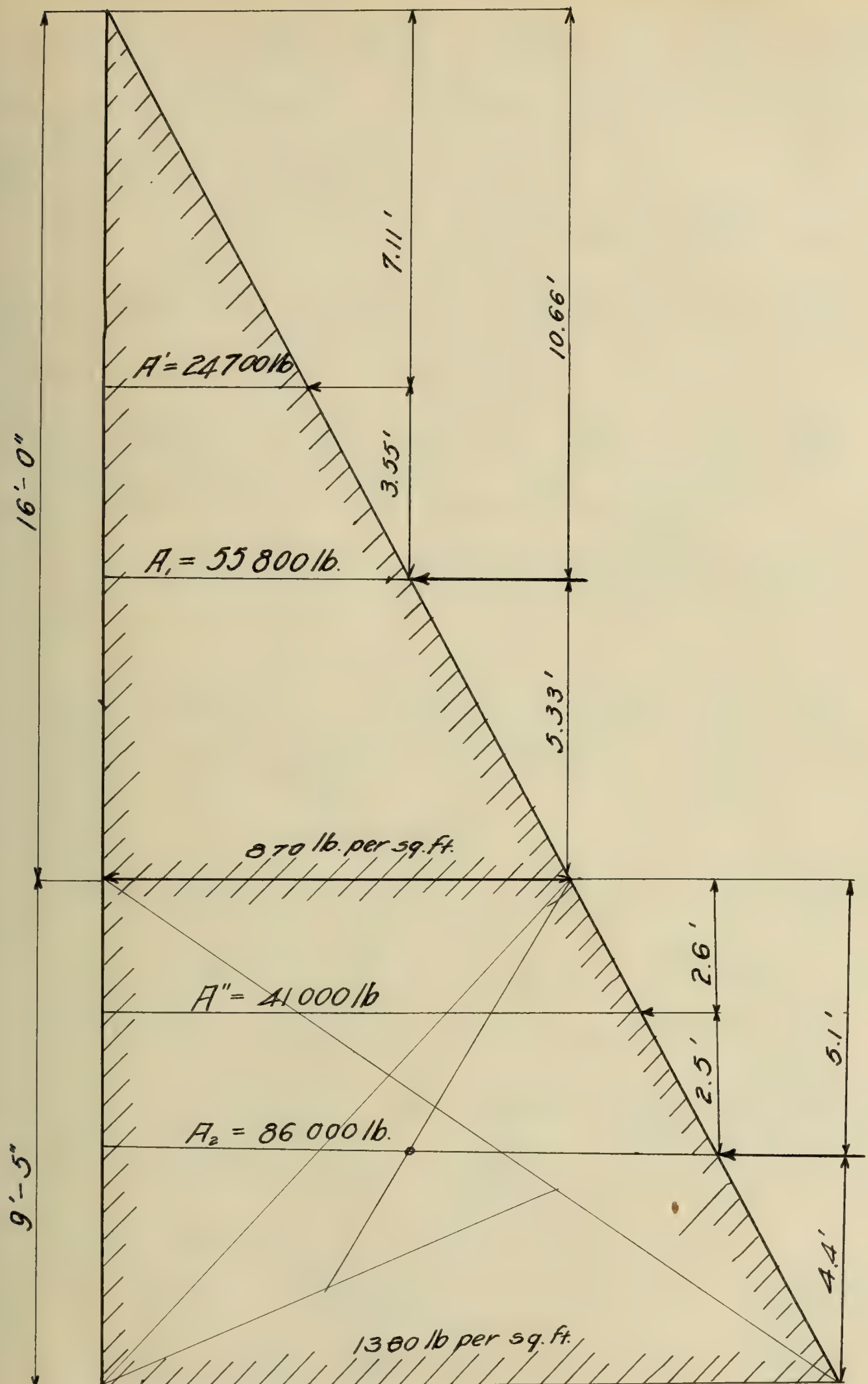


Fig. 13. Pressure Diagram for 22-ft. Bent.

By the use of the values shown on the diagram, the reactions at A and B, and the maximum bending moments in the panels are found.

The reaction at A is $55{,}800 \times \frac{5.33}{16.0} = 18{,}600$ lb. Using this reaction, the maximum moment in the first section is $18{,}600 \times 10.66 - 24{,}700 \times 3.55 = 10100$ lb.-ft, or 1,320,000 lb.-in. The reaction at B is $55{,}800 \times \frac{10.66}{16.0} + 86{,}000 \times \frac{4.3}{9.5} = 37{,}200 + 39{,}700 = 76{,}900$ lb. The maximum moment in the second panel is $39{,}700 \times 5.1 - 41{,}200 \times 2.5 = 99{,}000$ lb.-ft., or 1,190,000 lb. in.

The direct water load stresses in the members are found by graphic resolution. Fig. 14, and are given on the stress sheet.

The dead loads are estimated as follows: a footway is provided on the top of the dam, and its weight with whatever loads may come upon it is estimated at 200 lb. per linear ft.; the 3/8" facing-plate which is to be used weighs 15.3 lb. per sq. ft.

Dead load at A:

Footway;	$200 \times 8 =$	1,600 lb.
Face-plate;	$88 \times 15.3 =$	1,040
I-beam;	$40 \times 8 =$	320
AD;	$15 \times 11 =$	165
	Total =	3,130 lb.

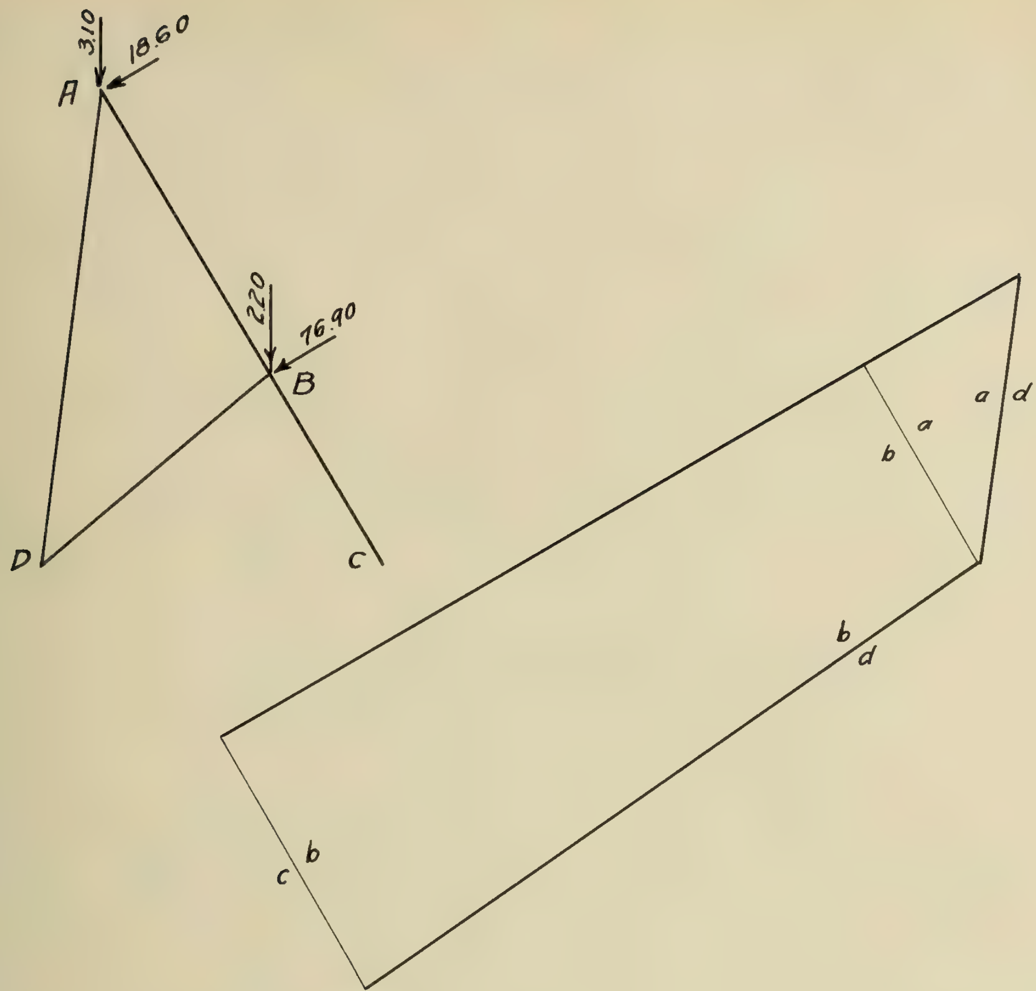


Fig. 14. Water Load
Scale: 1 in. = 20,000 lb.

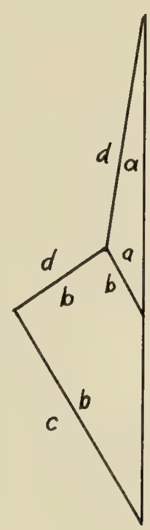


Fig. 15. Dead Load
Scale: 1 in. = 2,000 lb.

Dead load at B:

Face-plate; 15.3 x 8 x 12.75 =	1,560 lb.
I-beam; 40 x 12.75 =	510
BD; 20 x 6.6 =	132
Total =	2,200 lb.

The dead load stresses are shown on the stress sheet, p. 25. They are obtained by graphical resolution as illustrated in Fig. 15, p. 29.

DETERMINATION OF THE SECTION OF AB-BC

AB has the greater moment, p 28, and a section suitable for AB will be suitable for BC also. The direct stress in the member is equal to the sum of the water load and dead load stresses, or 24,000 − 700 = 23,300 lb. tension. The length of the member AB is 16 feet, or 192 inches, and the square of the length = 38,400. The maximum moment is 1,320,000 lb.-in. A section consisting of a 15"-42lb. I-beam is assumed. Its sectional area is 12.48 sq. in., and its moment of inertia is 442. The unit stress due to both bending and tension is

$$S = \frac{23{,}300}{12.48} + \frac{1{,}320{,}000 \times 7.5}{442 + \frac{23{,}300 \times 38{,}400}{290{,}000{,}000}}$$

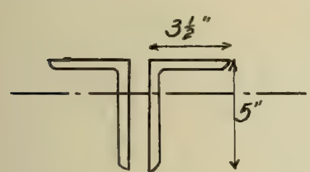
$$= 1{,}860 + 22{,}200 = 24{,}060 \text{ lb. per sq. in.}$$

The allowable stress is 25,000 lb. per sq. in. A 15"-42lb.

I-beam will therefore be used for AB and BC.

Determination of the Section of AD

The sway bracing will be put in as shown in Fig. 16. thus reducing the unsupported length of the column to 11 feet, or 132 inches. This member will be made up of two angles riveted back to back. The allowable unit stress is $24{,}000 - 110\frac{l}{r}$. A section consisting of two 5"x 3½"x 3/8" ∠s is assumed. They are placed 3/8 in. apart, the least radius of gyration being 1.33 in. for the section. $\frac{l}{r} = \frac{132}{1.33} = 99$. The allowable



stress is then $24{,}000 - 110 \times 99 = 13{,}100$ lb. per sq. in. The direct stress in the member due to water and dead load is $30{,}400 + 2{,}500 = 32{,}900$ lb. The required area is $32{,}900 \div 13{,}100 = 2.5$ sq. in. The area of the assumed section is 6.10 sq. in., but the requirement that $\frac{l}{r}$ shall be less than 100 would not be fulfilled if a smaller section was used, therefore the assumed section will be used for AD.

Determination of the Section of BD.

The unsupported length of the member is 13.4 ft.,

or 161 inches. The sum of the water and dead load stresses is 76,900 + 1,100 = 78,000 lb. A section is assumed consisting of 2 6"x4"x 3/8" Ls placed with the 6-inch legs back to back. The radius of gyration of this section is 1.67 inches, and $l/r = \frac{161}{1.67} = 96.5$. The allowable stress is 24,000 − 110 × 96.5 = 13,400 lb. per sq. in. The required area is 78,000 ÷ 13,400 = 5.8 sq. in. The area of the assumed section is 7.22 sq. in. This is the smallest section allowable for that length, and it will be used.

THE SWAY BRACING

Four 22-foot bents are located at each end of the dam. They are braced together in pairs as shown in Fig. 16. AA_3 consists of one $3\frac{1}{2}$"x 3"x 3/8" L riveted by the shorter leg. The diagonals are each one 3"x 3"x $\frac{5}{16}$" L. The bracing is attached to the columns by gusset-plates.

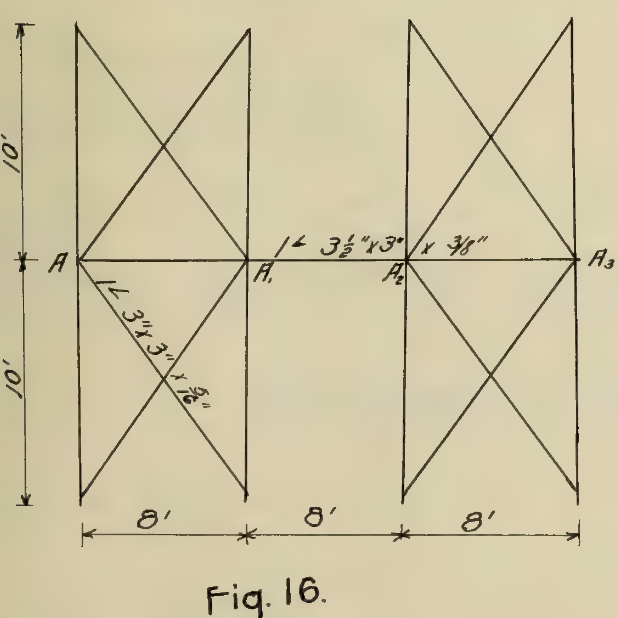


Fig. 16.

THE 42-FT. BENT

As described and illustrated on pages 8 and 9, ten bents, numbered 5, 6, 7, 8, 9, 28, 29, 30, 31, and 32, are all 42-feet high and of similar design. Fig. 17, p. 34 is a stress sheet for a 42-ft. bent. The slopes of the outside legs of the frame are the same as in the 22-ft. bent, and the general plans of the bents are similar. The length of the downstream leg is $42 \div \cos 8° = 42.5'$ ft., and the length of the upstream leg is $42 \div \cos 30°$ $= 48.5$ ft. The upstream leg of this bent is divided into three panels. The best lengths for these panels was found by trial to be 23 ft., 14 ft., and $11\frac{1}{2}$ ft., as shown on the stress sheet. The reactions at the panel points and the bending moments in the panels due to maximum water load are determined graphically as shown on page 35.

The water pressure at the bottom of the bent is $62.5 \times 42 = 2625$ lb. per sq. ft.

COMPUTATIONS OF BENDING MOMENTS

Panel AB:

The reaction at A is $114,900 \times \frac{7.7}{2\ 30} = 38,300$ lb.

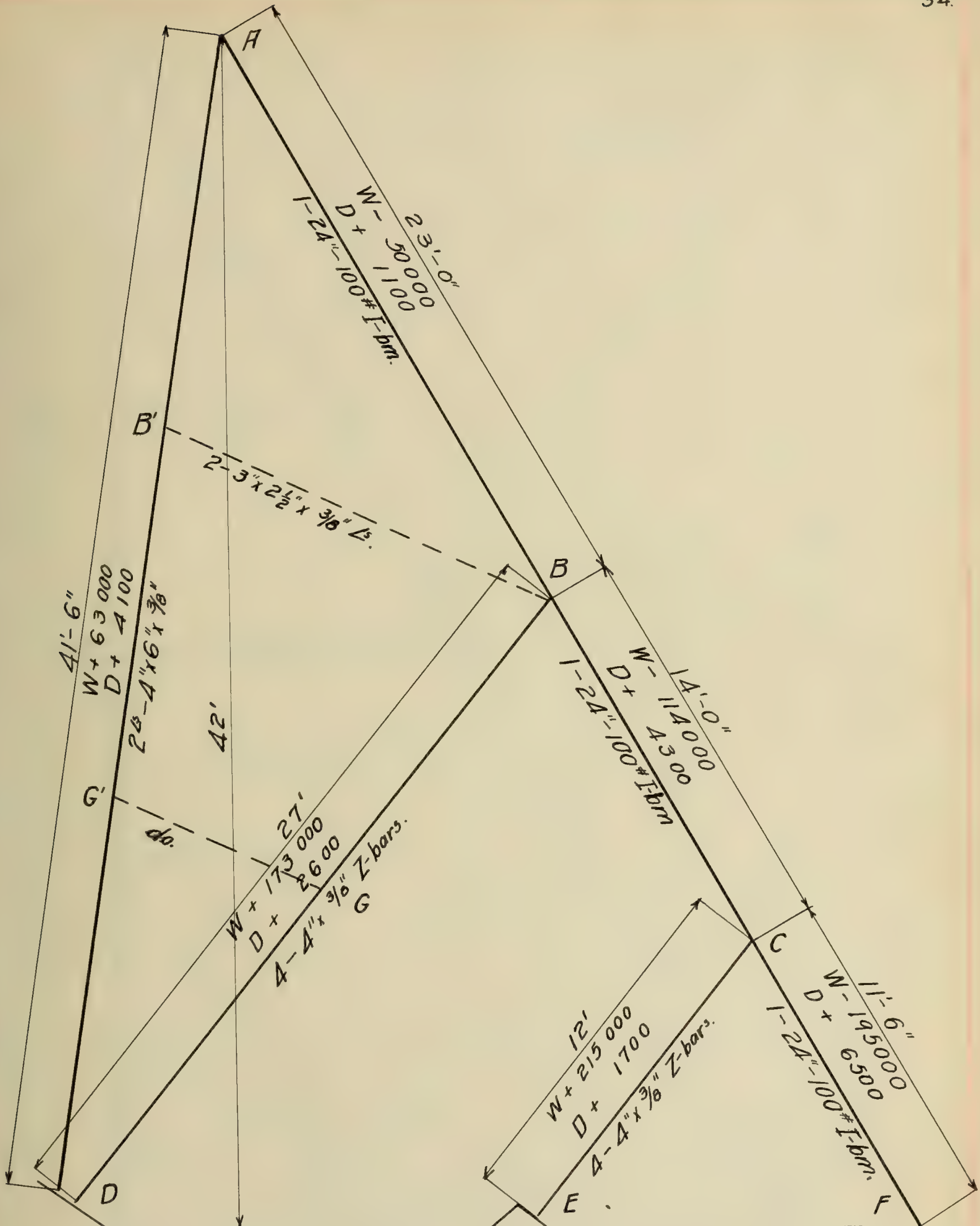


Fig. 17. Stress Sheet for 42-ft. Bent.

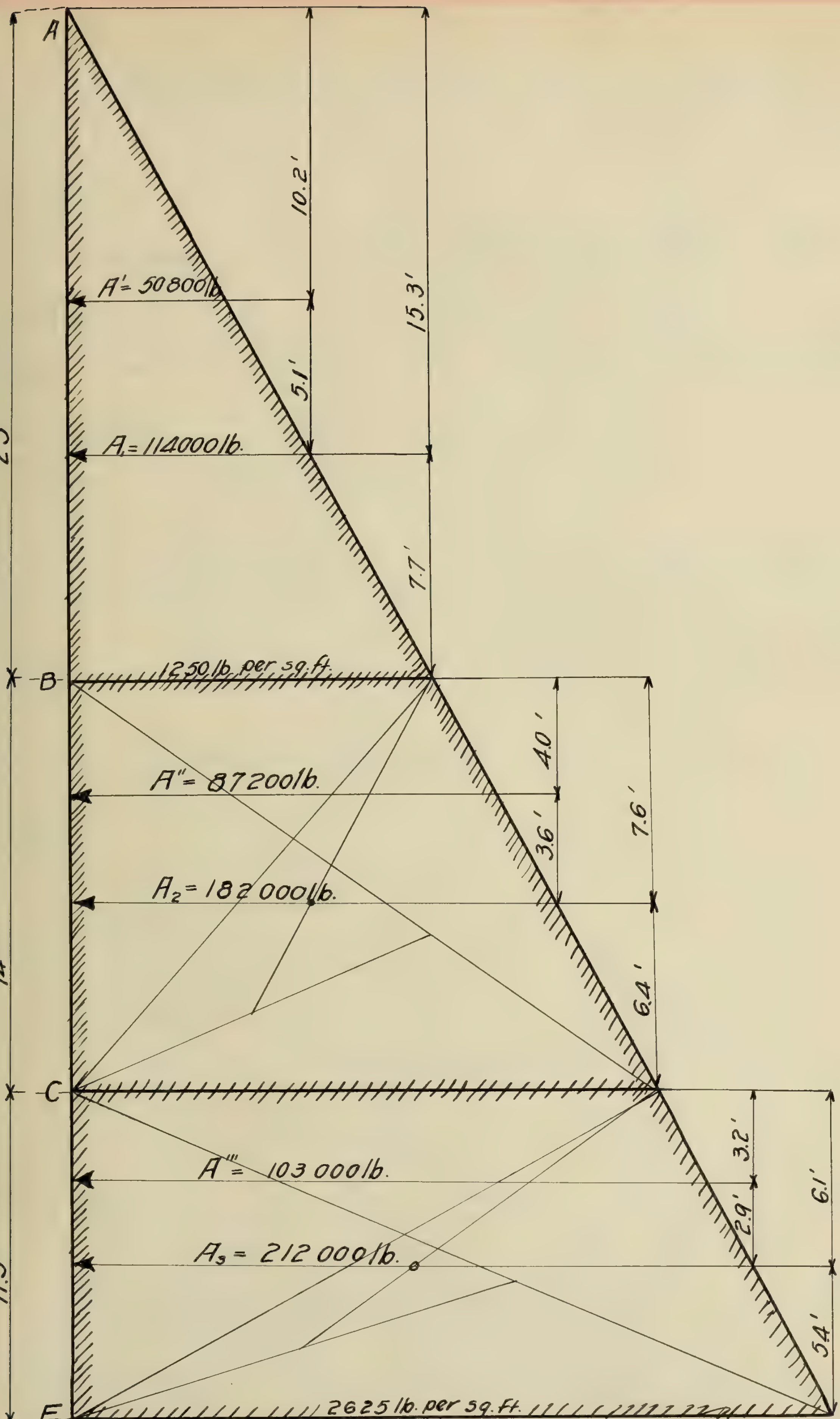


Fig. 18. Pressure Diagram for 42-ft Bent.

The maximum bending moment in the panel is
$38,300 \times 15.3 - 50,800 \times 5.1 = 32,600$ lb. ft., or 3,920,000 lb-in.

Panel B.C.–

The reaction at B is $114,000 \times \frac{15.3}{23.0} + 182,000 \times \frac{6.4}{14.0} = 76,600 +$
$84,200 = 160,800$ lb. The maximum moment in the second panel is $84,200 \times 7.6 - 87,200 \times 3.6 = 318,000$ lb.-ft.; or 3,820,000 lb-in.

Panel CF.–

The reaction at C is $182,000 \times \frac{7.6}{14.0} + 212,000 \frac{5.4}{11.5} = 97,800 +$
$101,000 = 198,800$ lb. The maximum bending moment is $101,000 \times 6.1 - 103,000 \times 2.9 = 317,000$ lb.-ft., or 3,800,000 lb-in.

ESTIMATED DEAD LOADS.

At A:

Footway; $200 \times 8 =$	1,600 lb.
Face-plate, $11.5 \times 8 \times 15.3 =$	1,410
I-beam, $100 \times 11.5 =$	1,150
Column, $21 \times 40 =$	840
Total	5,000 lb.

At B:

Face-plate, $18.5 \times 8 \times 15.3 =$	2,260 lb.
I-beam, $18.5 \times 100 =$	1,850
Column, $13.5 \times 50 =$	675
Total	4,790 lb.

At C:

Face-plate,	$18.5 \times 8 \times 15.3 =$	1,530 lb.
I-beam,	$12.7 \times 100 =$	1,270
Column,	$6 \times 60 =$	360
	Total	3,160 lb.

The dead- and water load stresses are determined graphically and separately on the following page. The stresses scaled from the diagram are recorded on the stress sheet, page 34.

DETERMINATION OF THE SECTION OF AB, BC & CF.

AB:

The direct stress in this member is the sum of the water and dead load stresses. The total stress is $50,000 - 1,100 =$ 48,900 lb. tension. The length of the panel is 23 ft., or 276 inches, and $l^2 = 76,500$. The maximum bending moment is 3,920,000 lb-in. A 24" 100 lb. I-beam is assumed for this member. Its sectional area is 29.41 sq.in. and the moment of inertia is 2,380.3 The stress in pounds per square inch produced by the direct and the bending forces is

$$S = \frac{48,900}{29.41} + \frac{3,920,000 \times 12}{2,380.3 + \dfrac{48,900 \times 76,500}{290,000,000}}$$

$$= 1,600 + 19,700 = 21,300 \text{ lb. per sq.in.}$$

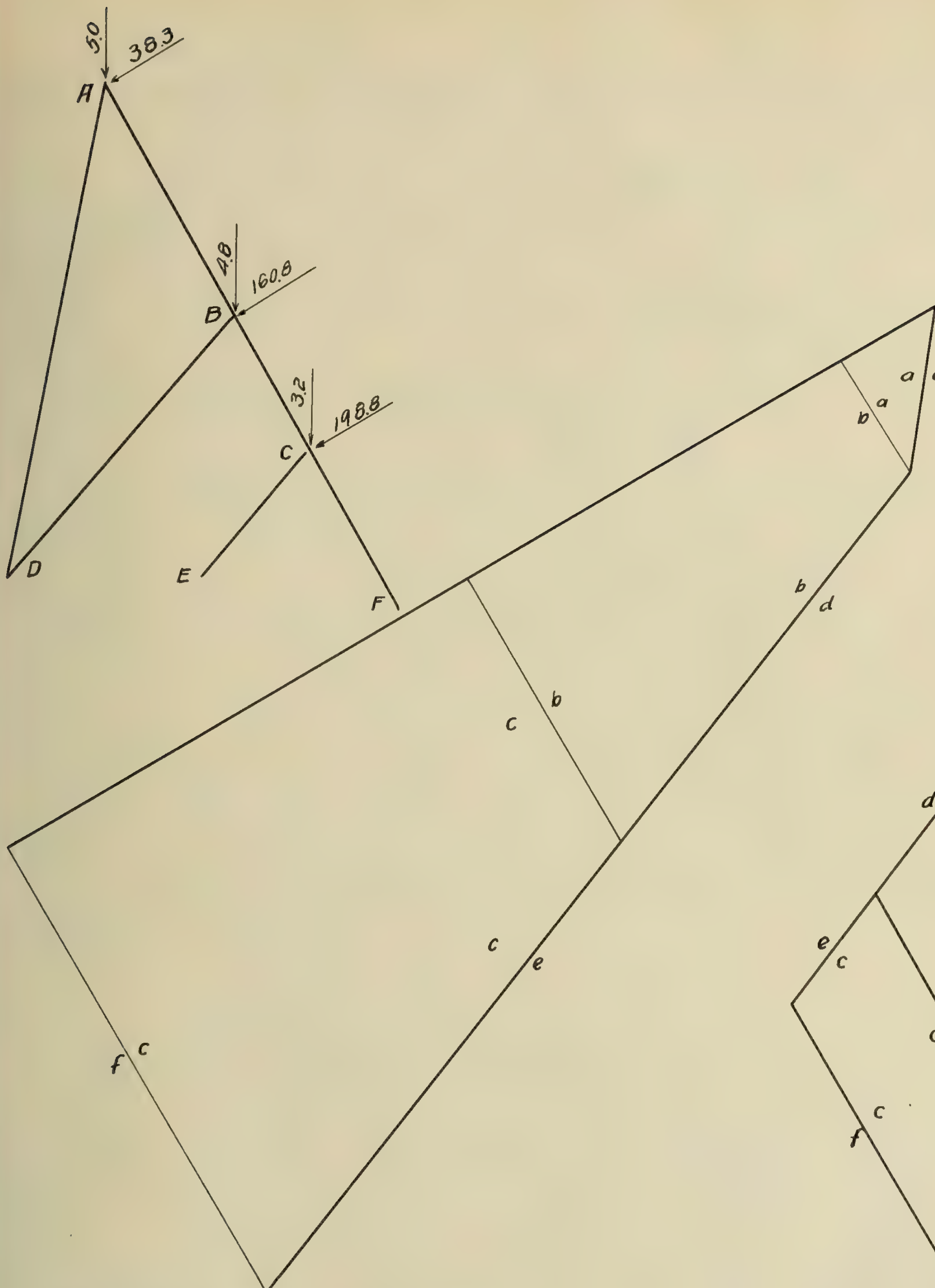


Fig. 19. Water Load Stresses
Scale: 1 in. = 60,000 lb

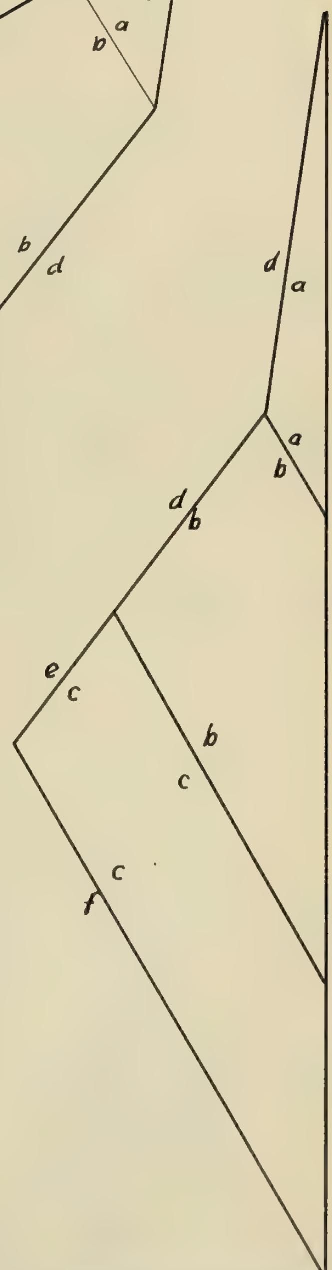


Fig. 20. Dead Load Stresses
Scale: 1 in. = 2,000 lb.

CF:

The dead and water load stresses amount to 195,000 − 6,500 = 188,500 lb. tension. The length is 138 inches, and $l^2 = 19{,}100$. The maximum moment is 3,800,000 lb-inches. The unit stress due to the direct and the bending force is

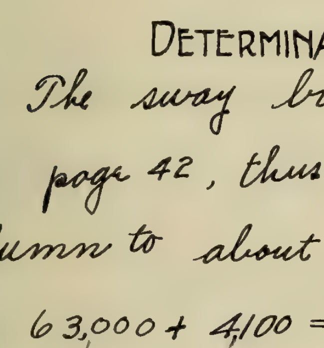
$$S = \frac{188,500}{29.41} + \frac{3,800,000 \times 12}{2,380.3 + \frac{188,500 \times 19,100}{290,000,000}}$$

$$= 6,400 + 19,000 = 25,400 \text{ lb. per sq. in.}$$

The span of this member will not be quite 11.5 ft., making the bending stress less than that computed, so that it will fall within the allowable limit of 25,000 lb. per sq. in. The stress in the middle panel will fall within the limit safely because of there being a lesser direct stress. A 24" 100 lb. I-beam will be used from A to D.

DETERMINATION OF THE SECTION OF AD.

1 The sway bracing will be used as shown in Fig. 21, page 42, thus limiting the unsupported length of the column to about 168 in. The sum of the direct stresses is 63,000 + 4,100 = 67,100 lb. A column consisting of two

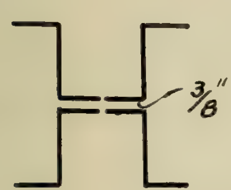


6" x 4" x 3/8" angles is assumed to support this stress. The radius of gyration of the section is 1.68 in., and $l/r = \frac{168}{1.68} = 100$. The

allowable stress is $24,000 - 110 \times 100 = 13,000$ lb. per sq. in. and the required sectional area is $67,100 \div 13,000 = 5.16$ sq. in. The area of the assumed section is 7.22 sq. in., and, since this is the smallest section giving the required radius of gyration, it will be used.

Determination of the Section of BD.

BD. will be braced as shown in Fig. 17; and its unsupported length will then be 174 inches. The water and dead load stresses in the member are 173,000 lb. and 2,600 lb., making a total of 175,600 lb. compression. A column is assumed consisting of four Z-bars, 4" x 3/8".



The radius of gyration of this section is 2.57 in., and $l/r = 65.3$. The allowable stress is $24000 - 110 \times 65.3 = 16,800$ lb. per. sq. in. The required area is $175,600 \div 16,800 = 10.45$ sq. in. The assumed section, giving an area of 14.64 sq. in., will be used.

Determination of the Section of CE.

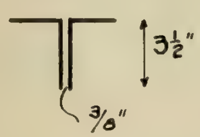
The length of this member is 144 inches. The sum of the direct stresses is $215,000 + 1,700 = 216,700$ lb. compression. To withstand this, a section consisting of four 4 in x 3/8 in Z-bars is assumed. The radius of gyration is 2.57 in. $l/r = 56$. The allowable stress in

the member is $24{,}000 - 110 \times 56 = 17{,}800$ lb. per sq. in. The required area is $216{,}700 \div 17{,}800 = 12.10$ sq. in. The assumed section, giving an area of 14.64 sq. in., will be used.

SWAY BRACING FOR 42-FT. BENTS.

The manner in which the 42-ft bents are braced together is illustrated in Fig. 21. The struts, AA and BB, are each made up of two $3\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{3}{8}''$ ∟s placed with the longer legs back to back. Each diagonal is composed of one $3'' \times 3'' \times \frac{3}{8}''$ ∟.

In addition to the bracing shown in Fig. 21, the columns BD and AD are braced by BB′ and GG′ as shown in Fig. 17, and the points G are connected by struts in the same manner as are points G′ and B′. All these struts are of the same section being two angles $3\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{3}{8}''$ with the longer legs back to back as shown in sketch. Each



member in the bracing system is fastened to its gusset plates by two 7/8 in. rivets in each end. The gusset plates are connected to the columns by from three to five or six rivets apiece.

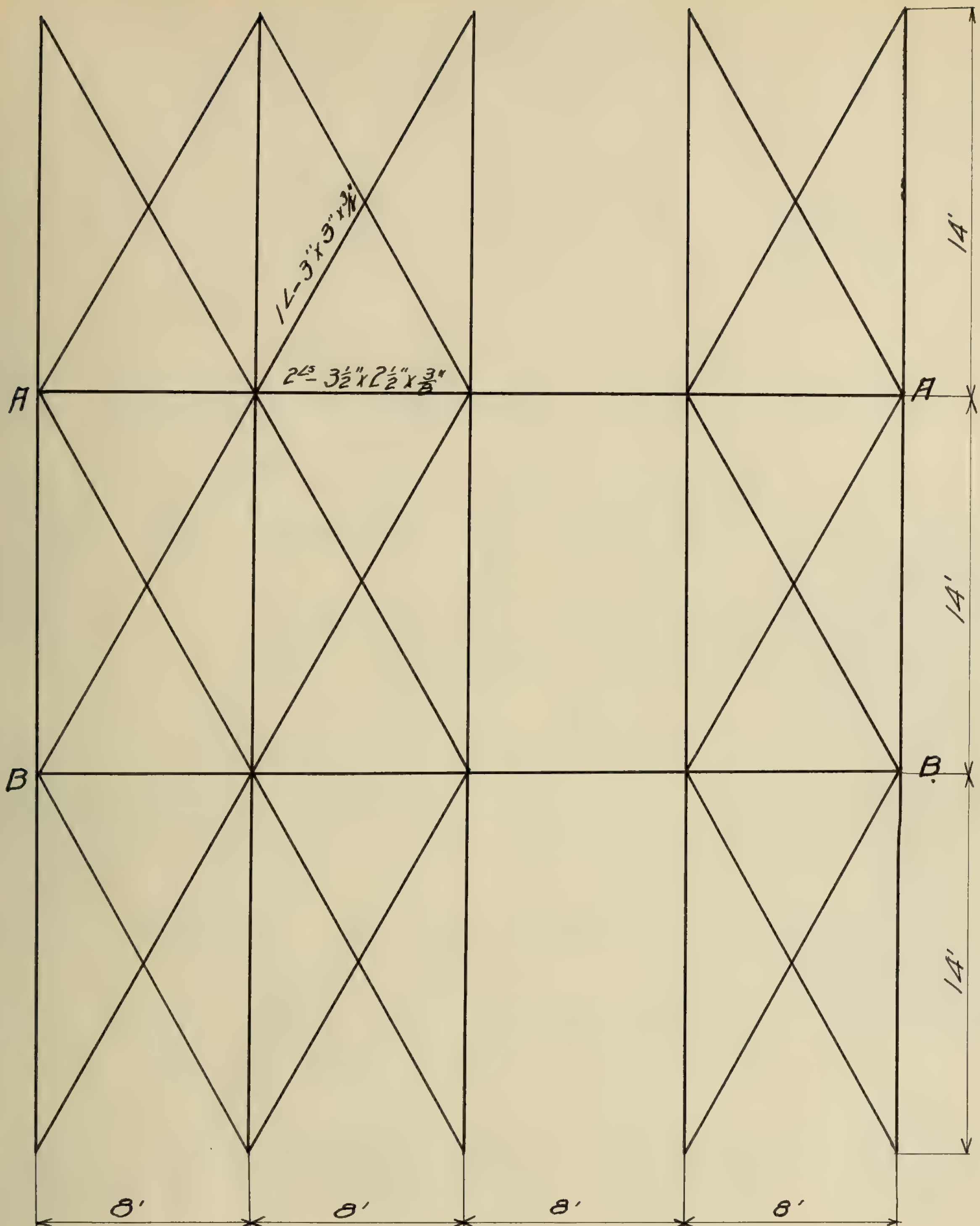


Fig. 21. Bracing for 42-ft. Bent.

THE 58-FT. BENT

From the general lay out as shown on pages 8 and 9, it will be seen that the bents numbered 10, 11, 12, 13, 22, 23, 24, 25. 26 and 27 are each 58 feet high. Fig. 22 is a stress sheet of one of these bents. The length of the downstream leg is $58 \times \cos 8^\circ = 58.8$ ft. and the length of the upstream leg is $58 \times \cos 30^\circ = 67$ ft. The upstream leg of the bent is divided into five panels measuring in length respectively 23, 14, 11, 10, and 9 feet. Since the first two panels are the same length as the corresponding ones in the 42-ft bent, their end reactions and maximum bending moments are the same. The reactions and moments in the three remaining panels are found with the aid of the graphical analysis shown in Fig. 23

The water pressure per square foot at the bottom of the bent is $62.5 \times 58 = 3{,}625$ lbs.

COMPUTATION OF BENDING MOMENTS.

Panel CD:

The reaction at C is $182{,}000 \times \frac{26}{14.0} + 202{,}000 \times \frac{5.2}{11.0} = 99{,}000 + 95{,}400 = 194{,}400$ lb. The maximum bending mom-

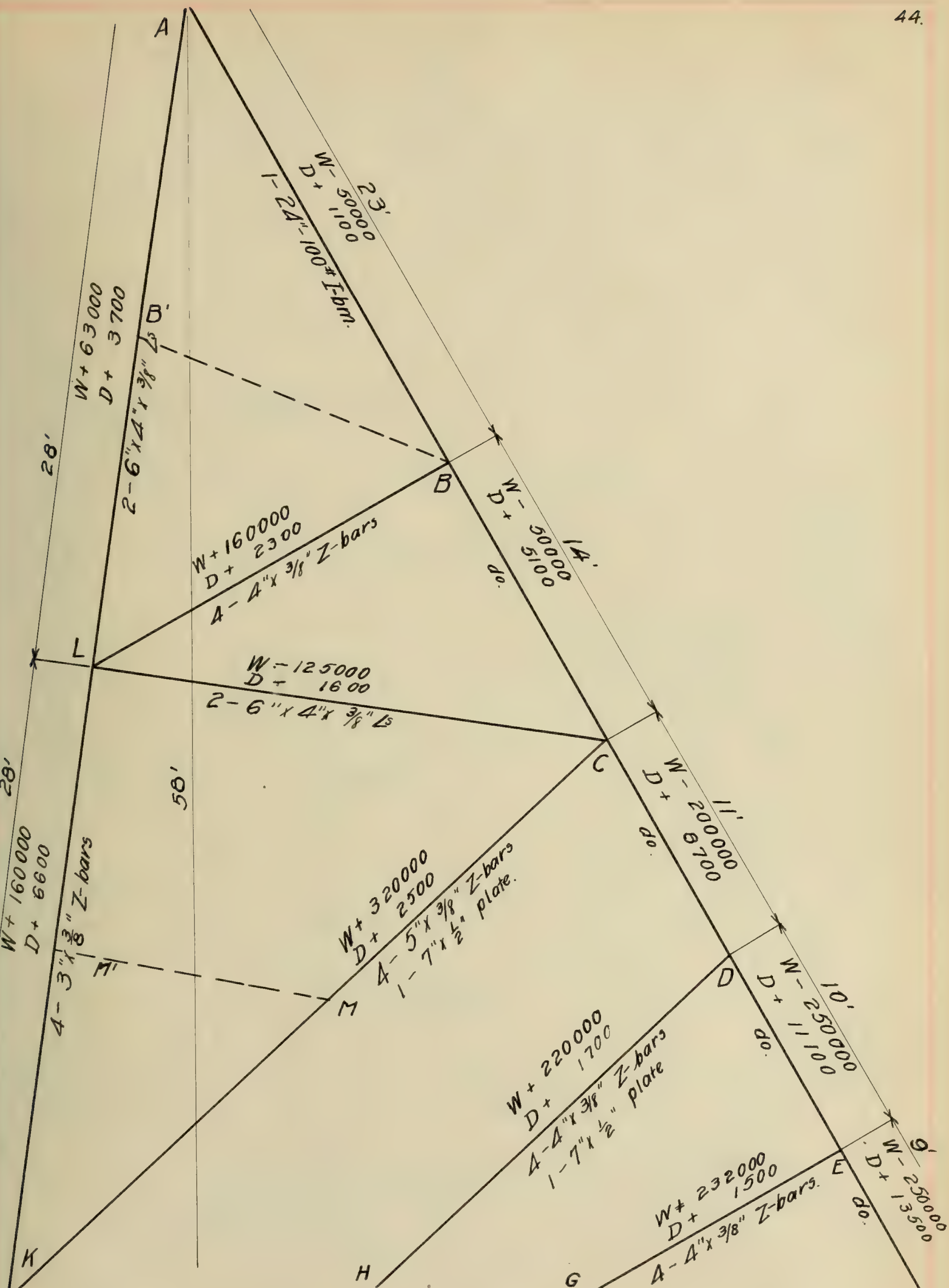


Fig. 22. Stress Sheet for 58-ft. Bent.

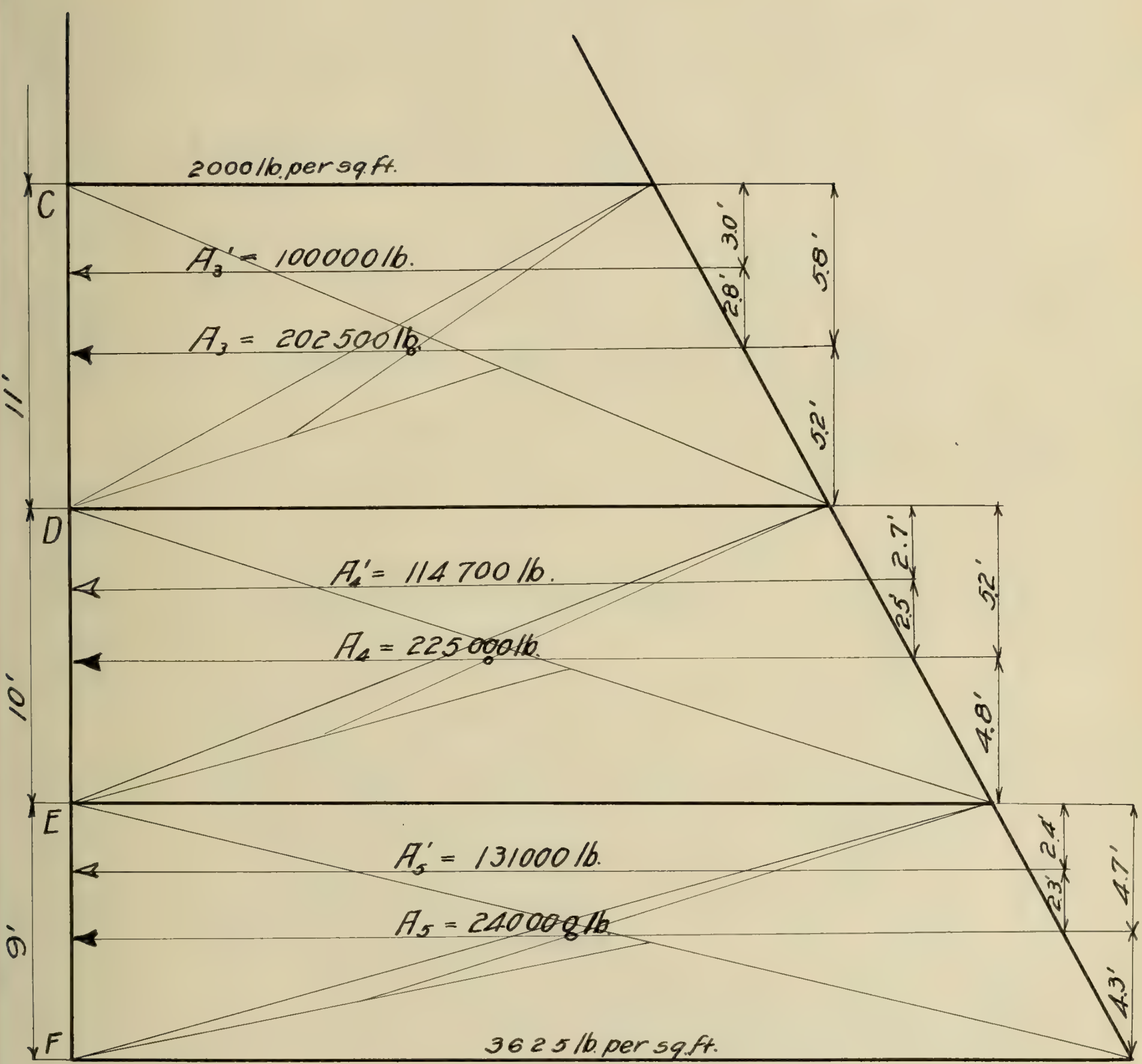


Fig. 23. Pressure Diagram for 58 ft. Bent

ent in the panel is $95,400 \times 5.8 - 100,000 \times 2.8 = 277,000$ lb.-feet, or 3,320,000 lb.-inches.

Panel DE:

The reaction at D is $202,000 \times \frac{5.8}{11.0} + 225,000 \times \frac{4.8}{10.0} =$ $106,500 + 10,8000 = 214,500$ lb. The maximum bending moment in the panel is $108,000 \times 5.2 - 114,700 \times 2.5 =$ 270,000 ~~lb.~~-feet, or 324,000 lb.-inches.

Panel EF:

The reaction at E is $225,000 \times \frac{5.2}{10.0} + 240,000 \times \frac{4.3}{9.0} = 117,000$ $+ 114,500 = 231,500$ lb. The maximum bending moment is $114,500 \times 4.7 - 131,000 \times 2.3 = 238,000$ lb-feet or 2,860,000 lb.-inches.

ESTIMATED DEAD LOADS.

At A:

Footway, $200 \times 8 =$	1,600 lb.
Face-plate, $11,5 \times 8 \times 15.3 =$	1,400
I-beam, $11.5 \times 100 =$	1,150
Column, $14 \times 30 =$	420
Total	4,580 lb.

At B:

Face-plate, $18.5 \times 8 \times 15.3 =$	2,260 lb.
I-beam, $18.5 \times 100 =$	1,850
Column, $9 \times 55 =$	500
Total	4,610 lb.

At C:

Face-plate,	12.5 x 8 x 15.3 =	1,530 lb.
I-beam,	12.5 x 100 =	1,250
CK,	18 x 100 =	1,800
CL,	12 x 20 =	240
	Total	4,820 lb.

At D:

Face-plate,	10.5 x 8 x 15.3 =	1,290 lb.
I-beam,	10.5 x 100 =	1,050
Column,	11 x 80 =	880.
	Total	3,220 lb.

At E:

Face-plate,	9.5 x 8 x 15.3 =	1,160 lb.
I-beam,	9.5 x 100 =	950
Column,	7 x 80 =	560
	Total	2,670 lb.

At L:

Three columns, 28 x 30 + 9 x 55 + 12 x 20 =

Total 1,575 lb.

The stresses in the members due to dead and water loads are found by graphical analysis, as shown on the following page.

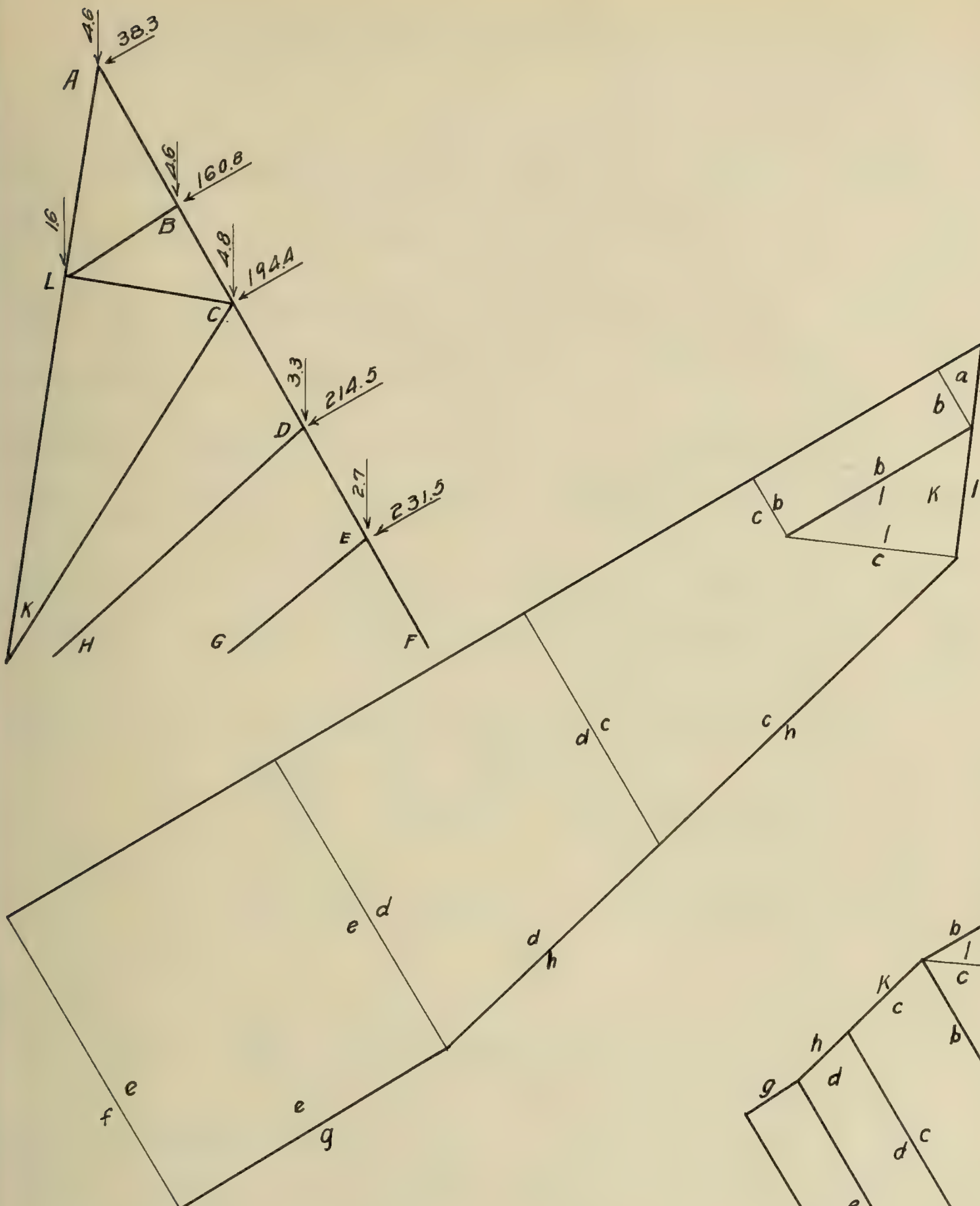


Fig. 24. WATER LOADS
Scale: 1 in = 100,000 lb.

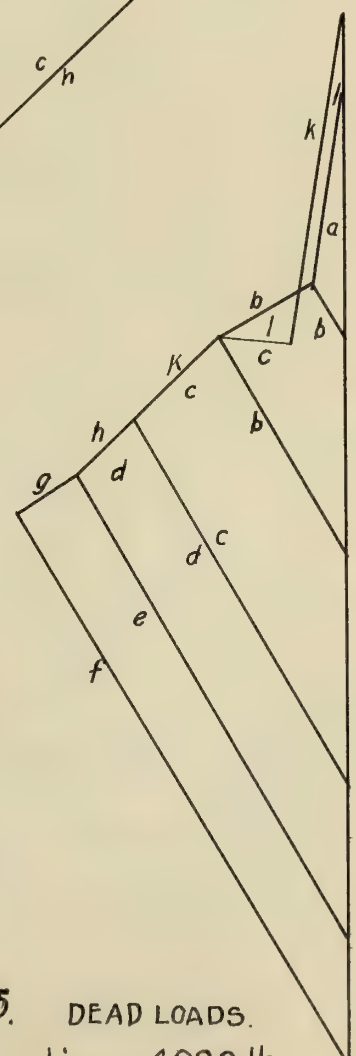


Fig. 25. DEAD LOADS.
Scale: 1 in = 4,000 lb.

DETERMINATION OF THE SECTION OF CD, DE & EF.

CD:

The direct water load stress is − 200,000 lb., and the dead load stress is + 8,700 lb.; the stress designed for being their sum, or 191,300 lb. tension. The length of the member is 132 inches and $l^2 = 17{,}500$. The maximum bending moment is 3,320,000 lb.-inches. A section consisting of a 24" 100 lb. I-beam is assumed. Its sectional area is 29.41 sq. in., and moment of inertia is 2,380.3. The unit stress produced in this beam by combined direct and bending forces is

$$S = \frac{191{,}300}{29.41} \mp \frac{3320000 \times 12}{2{,}380.3 + \frac{17{,}500 \times 191{,}300}{290{,}000{,}000}}$$

$$= 6{,}500 + 16{,}700 = 23{,}200 \text{ lb. per sq. in.}$$

DE:

The direct water load stress in this panel is −250,000 lb., and the dead load stress is +11,100 lb.; combined they produce a tensile stress of 238,900 lb. The panel is 120 inches long and $l^2 = 14{,}400$. The maximum bending moment is 3,240,000 lb.-inches. Assuming a 24" 100 lb. I-beam, the sectional area and moment of inertia being as given above, the unit stress produced by tension and bending is

$$S = \frac{238{,}900}{29.41} + \frac{3{,}240{,}000 \times 12}{2380.3 + \frac{14400 \times 238{,}900}{290{,}000{,}000}}$$

$= 8{,}150 + 16{,}300 = 24{,}450$ lb. per sq. in.

Since both the direct stress and the bending moment in EF are slightly smaller than they are in DE, it will not be necessary to compute the unit stress in EF as it is evident that it will fall within the allowable limit if a 24" 100 lb. I-beam be used. This size will be used for all the panels of the upstream leg.

DETERMINATION OF THE SECTION OF AL.

The sway bracing, as shown on page 54, will limit the unsupported length of this column to about 168 inches. The sum of the direct dead and water load stresses is $63{,}000 + 3{,}700 = 66{,}700$ lb. compression. To withstand this stress a column is assumed consisting of two 6"x 4"x 3/8" ∠s placed with the longer legs back to back and ½ inch apart. The radius of gyration of the section is 1.68 inches, and $l/r = 100$. The allowable unit stress is $24{,}000 \div 110 \times 100$ $= 13{,}000$ lbs. per sq. in. The required area is $66{,}700 \div 13{,}000 = 5.13$ sq. in. The assumed section gives an area of 7.22 sq. in., but, since a smaller section

would give too great a value for l/r, the assumed section will be used.

DETERMINATION OF THE SECTION OF LK.

The unsupported length is about 168 inches. The sum of the dead and water loads is 6,600 + 160,000 = 166,600 lb. compression. To support this, a column consisting of four 3"x 3/8" Z-bars and one 6"x 3/8" plate is assumed. The radius of gyration for the section is 1.88

inches, and l/r = 89. The allowable unit stress is 24,000 − 110 x 89 = 14,200 lb. per sq. in. The required area is 166,600 ÷ 14,200 = 11.75 sq. in. Since the area of the assumed section is 13.69 sq. in., it will be considered satisfactory.

DETERMINATION OF THE SECTION OF BL

The sum of the direct stresses in the member is 160,000 + 2,300 = 162,300 lb. compression, and the length of the member is 216 inches. A column is assumed consisting of four 4"x 3/8" Z-bars. The radius of gyration is 2.56 in. and l/r = 84. The allowable unit stress is 24,000 − 110 x 84 = 14,700 lb. per sq. in. The area required is 162,300 ÷ 14,700 = 11.00 sq. in. The assumed sec-

tion has an area of 14.64 sq. in. which is rather heavy; but since a lighter section would give too great a value for l/r, the above section will be used.

Determination of the Section of CK.

The unsupported length of this member is 180 in.; and the sum of the direct stresses is 320,000 + 2,500 = 322,500 lb. compression. A column is assumed consisting of four 5" x 3/8" Z-bars and a 7" x ½" plate. The radius of gyration is 3.13 inches and l/r = 58. The allowable unit stress is 24,000 − 110 × 58 = 17,660 lb. per sq. in. The section assumed gives an area of 19.90 sq. in., and since the required area is 322,500 ÷ 17,660 = 18.25 sq. in., the assumed section will be used.

Determination of the Section of DH.

This member is 216 inches long and loaded with 220,000 + 1,700 = 221,700 lb. compression. The assumed section consists of four 4" x 3/8" Z-bars and a 7" x 3/8" plate. The allowable stress is 24,000 − 110 × 84 = 14,700 lb. per sq. in. The required area is 221,700 ÷ 14,700 = 15.00 sq. in.; and the assumed section, which gives 17.10 sq. in., will be used.

Determination of the Section of EG

The length is 144 inches, and the direct stress is

232,000 + 1,500 = 233,500 lb. compression. A column is assumed consisting of four 4" x 3/8" Z-bars. The radius of gyration is 2.57 inches and $l/r = 56$. The allowable unit stress is 24,000 − 110 x 56 = 17,840 lb. per sq. in. The sectional area required is 233,500 ÷ 17,840 = 13.1 sq. in. The assumed section gives 14.64 sq. in, and will be used.

Determination of the Section of CL.

The total stress is 125,000 + 1,600 = 126,600 lb. tension. The allowable unit stress is 25,000 lb. per sq. in. 126,600. ÷ 25,000 = 5.08 sq. in., the required sectional area. A section is assumed consisting of two 6" x 3½" x 3/8" Ls riveted to gusset plates by the longer legs. The gross area of this section is 2 x 3.42 = 6.84 sq. in. Taking out two rivet holes leaves 6.84 − .75 = 6.09 sq in., net area. The assumed section is sufficient and will be used.

The Sway Bracing

The bracing between columns AK is shown on page 54. The points M, (Fig. 22), are connected by struts similar to those connecting M'. M'M and B'B are of the same section. Each of these struts consists of two 3½" x 2½" x 3/8" Ls with the 3½" legs back to back. Each diagonal consists of one 3" x 3" x 3/8" L.

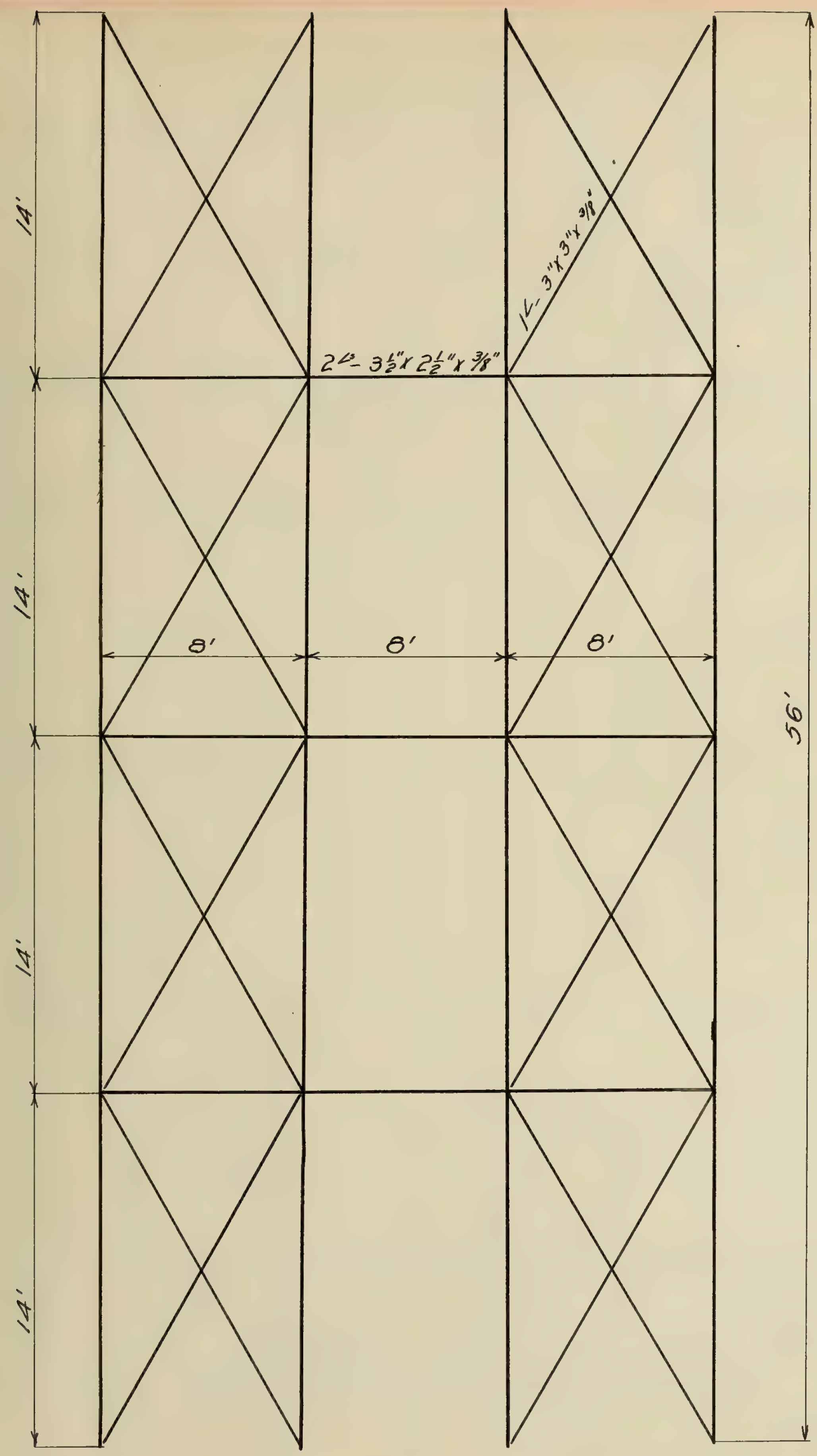


Fig. 26. Bracing for 58 ft. Bents.

THE 78FT BENT

As shown on pages 8 and 9, the bents numbered 14, 15, 16, 17, 18, 19, 20 and 21 are the highest in the dam, each being 78 feet high. Fig. 27. is a stress sheet of one of these bents. The length of the downstream leg is 78 x cos 8° = 78.8 ft. The length of the upstream leg is 78 x cos 30° = 90 ft. The upstream leg is divided into eight panels measuring respectively 23, 14, 11, 10, 9, 8.25, 7.75, and 7 ft. Since the first five panels are the same length as those in the 58-ft bent, the bending moments in these panels will be the same. The direct tensile stresses are nearly the same, and there-fore, without investigation, the same section may be used for these panels as for the 58-ft bent. The moments and reactions in the three remaining panels are found with the aid of the graphical analysis shown in Fig. 28.

The unit pressure at the bottom of the dam is 62.5 x 78 = 4,870 lb. per sq. ft.

COMPUTATION OF MOMENTS AND REACTIONS.

The reactions at the panel points above E will be the same as those in the 58-ft. bent.

E G:

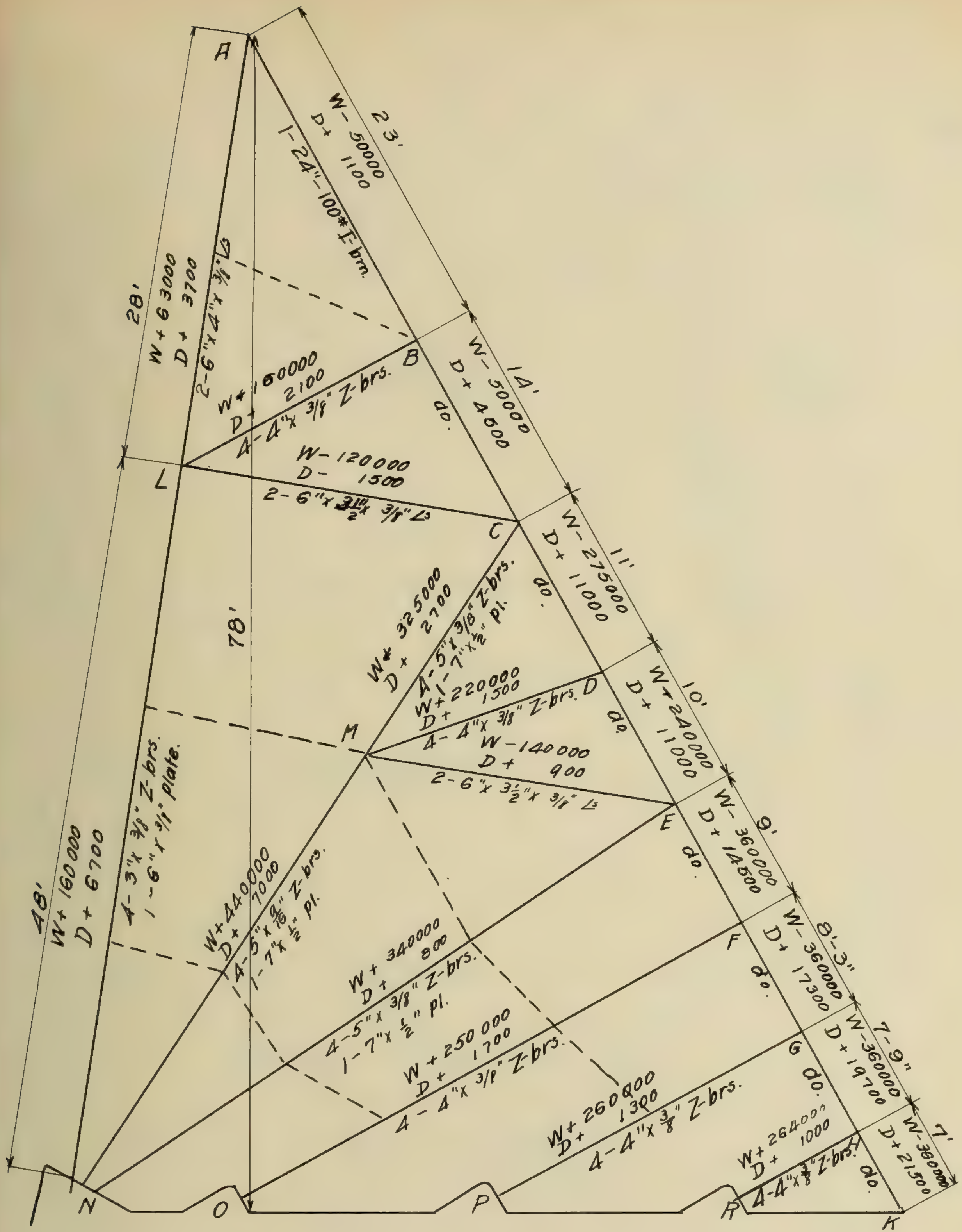


Fig. 27. Stress Sheet for 78-ft. Bent.

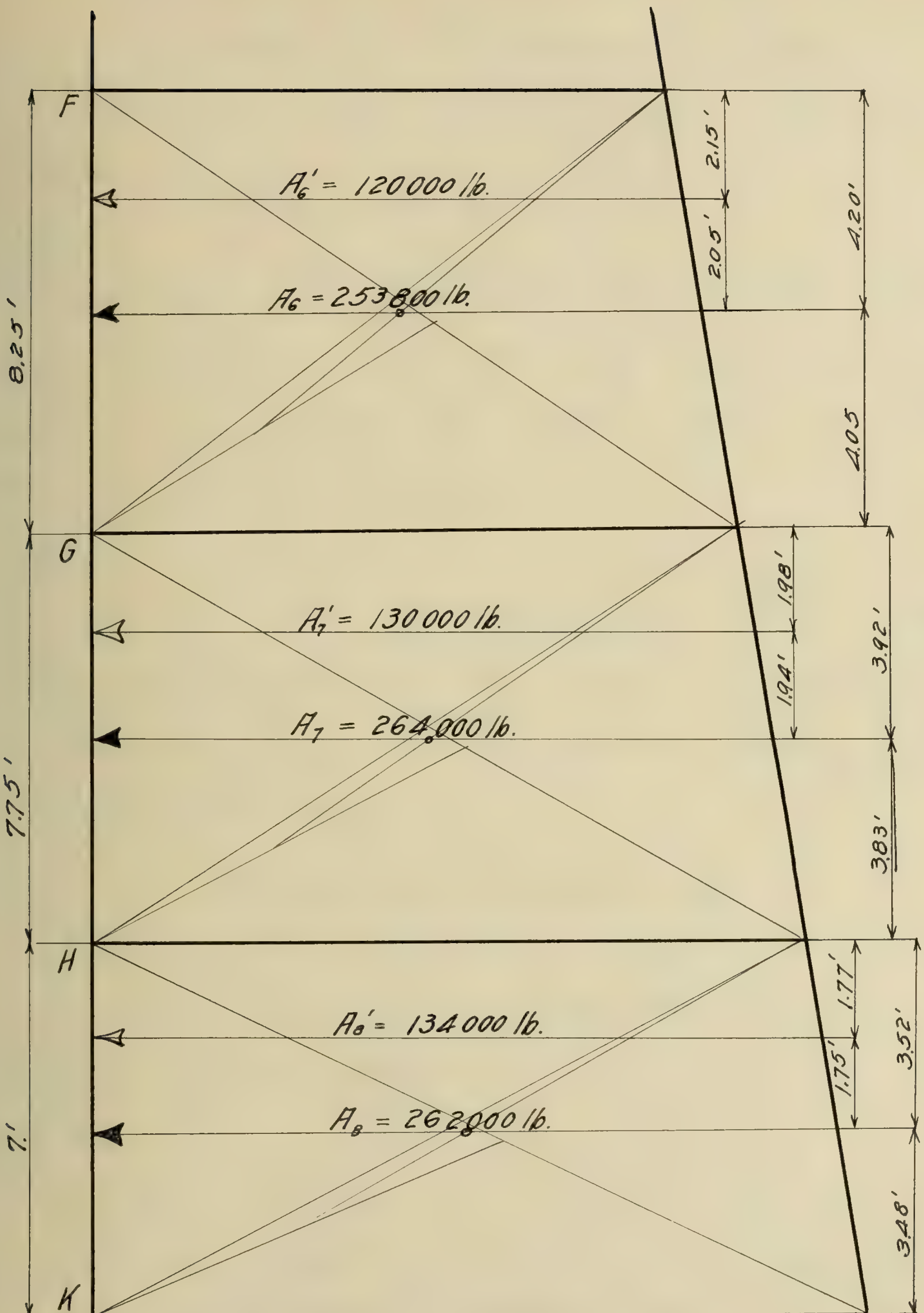


Fig. 28. Pressure Diagram for 78-ft. Bent.

The reaction at E is $240,000 \times \frac{4.7}{9.0} + 253,000 \times \frac{4.05}{8.25} = 125,000 + 12,4000 = 249,000$ lb. The maximum bending moment is $124,000 \times 4.20 - 120,000 \times 2.05 = 276,000$ lb-feet, or 3,310,000 lb-inches.

GH:

The reaction at G is $253,000 \times \frac{4.20}{8.25} + 264,000 \times \frac{3.83}{7.75} = 129,000 + 130,500 = 259,500$ lb. The maximum bending moment is $130,500 \times 3.92 - 130,000 \times 1.94 = 25,900$ lb-feet, or 31,000,000 lb.-inches.

HK:

The reaction at H is $264,000 \times \frac{3.92}{7.75} + 262,000 \times \frac{3.48}{7.00} = 134,000 + 130,000 = 264,000$ lb. The maximum bending moment is $130,000 \times 3.52 - 134,000 \times 1.75 = 226,500$ lb-feet, or 2,715,000 lb.-inches.

Estimate of the Dead Loads

The dead loads which are considered as concentrated at A, B, C, and L, are approximately the same as those computed for the 58-ft. bent, and need not be re-computed.

At D:

Face-plate,	$10.5 \times 8 \times 15.3 =$	1,290 lb.
I-beam,	$10.5 \times 100 =$	1,050
Column,	$8 \times 70 =$	560
	Total	2,900

At E:

Face-plate, 9.5 x 8 x 15.3 =	1,160 lb.
I-beam, 9.5 x 1,000 =	950
EN, 7.5 x 100 =	750
EM, 10 x 30 =	300
Total	3,160 lb.

At F:

Face-plate, 8.6 x 8 x 15.3 =	1,050 lb.
I-beam, 8.6 x 100 =	860
Column, 19 x 80 =	1,520
Total	3,430 lb.

At G: F-

Face-plate, 8 x 8 x 15.3 =	980 lb.
I-beam, 8 x 100 =	800
Column, 12 x 80 =	960
Total	2,740 lb.

At H:

Face-plate, 7.4 x 8 x 15.3 =	900 lb.
I-beam, 7.4 x 100 =	740
Column, 6 x 80 =	480
Total	2,120 lb.

At M:

Column, 8 x 70 =	560 lb.

Column, $9 \times 90 =$		810 lb.
" , $16 \times 100 =$		1,600
Tension member. $10 \times 30 =$		300
	Total,	3,270 lb.

The dead and water load stresses in the members of the bent are determined graphically in Figs. 29 and 30, page 61.

DETERMINATION OF THE SECTION FOR E-K

Of the four panels, EF, FG, GH, and HK, FG has the greatest moment, while the direct stress is approximately the same in all.

The direct stress in this member caused by combined water and dead loads is 360,000 - 17,300 = 342,700 lb. tension. The length of the member is 99 inches, and $l^2 = 9{,}830$. The maximum bending moment is 3,310,000 lb.-in. Assuming a 24" 100 lb. I-beam, the unit stress caused by tensile and bending forces is

$$S = \frac{342{,}700}{29.41} + \frac{3{,}310{,}000 \times 12}{2{,}380.3 + \frac{9{,}830 \times 342{,}700}{290{,}000{,}000}}$$

$$= 11{,}700 + 16{,}300 = 28{,}000 \text{ lb. per sq. in.}$$

The gusset plates which connect the columns to the beams will brace the beams and reduce the bending moment so that the actual stress will not be

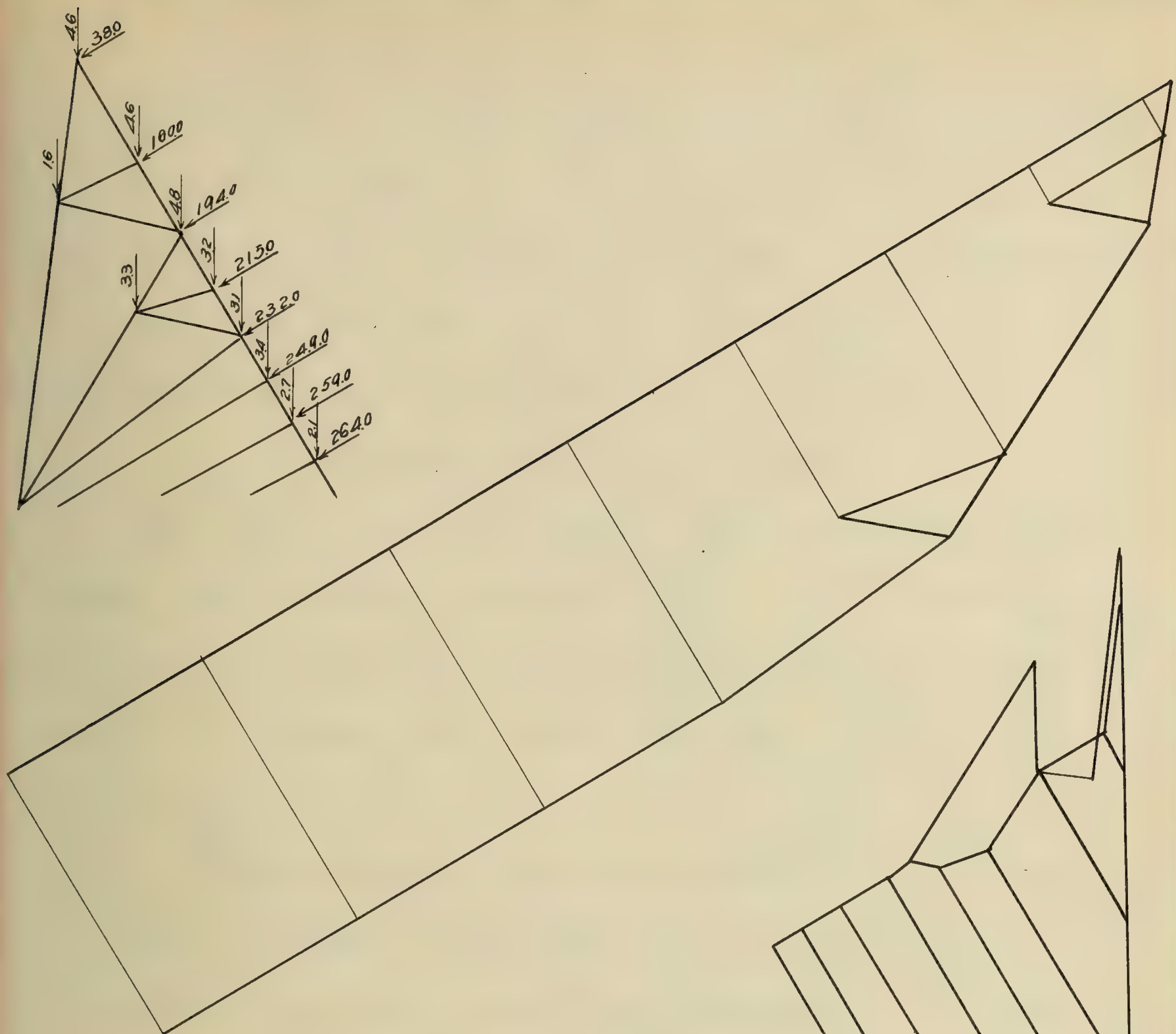


Fig. 29. WATER LOAD.
Scale: 1 in. = 200,000 lb.

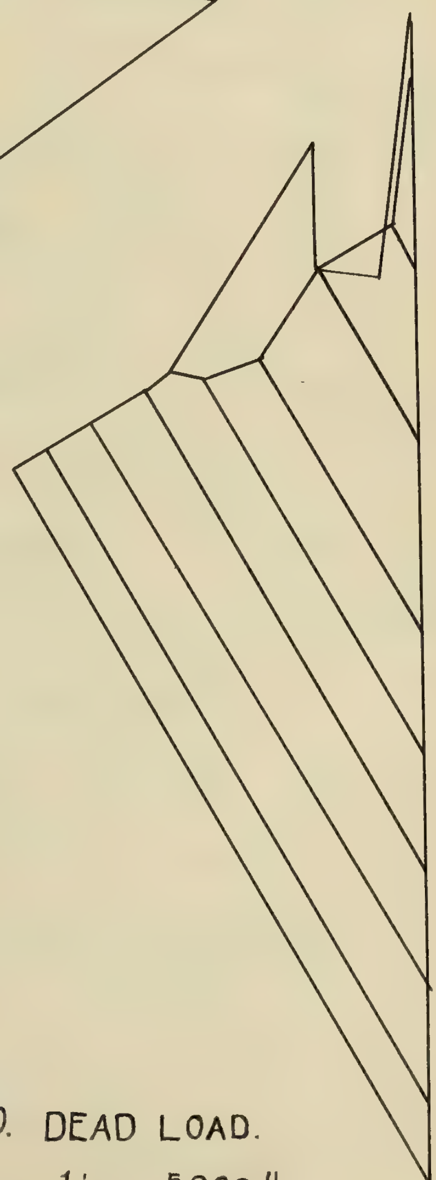


Fig. 30. DEAD LOAD.
Scale: 1 in. = 5,000 lb.

above the allowable limit. Since the moments in all the other panels are less than in the one just considered, the stresses in them will also be less, and the section used in the panel just investigated will be sufficent for each of the other panels. A 24" 100 lb. I-beam will be used for each panel.

MEMBERS AL, LK, BL, LC, AND CM.

All these members have approximately the same lengths and stresses as those found in the corresponding members in the 58-ft bent. The sections for these will therefore be made the same in this as in the 58-ft. bent.

DETERMINATION OF THE SECTION OF DM

This member is 180 inches long, and the direct stress is 222,000 + 1,500 = 221,500 lb. compression. A column is assumed which consists of four 4" x 3/8" Z-bars latticed. The radius of gyration is 2.57 inches, and l/r = 70. The allowable unit stress is 24,000 − 110 x 70 = 16,300 lb. per sq. in. The area required is 221,500 ÷ 16,300 = 13.50 sq. in. The assumed section gives an area of 14.64 sq. in., and will be used.

DETERMINATION OF THE SECTION OF EM

The sum of the direct stresses is 140,000 − 900 = 139,100

lb. tension. The area required is 139,100 ÷ 25,000 = 5.57 sq. in. A section consisting of two 6" x 3½" x 3/8" ∠s is assumed. The gross area of the section is 2 x 3.42 = 6.84 sq. in. The net area is the gross area less two rivet holes, or 6.84 − .75 = 6.09 sq. in. The above section will be used.

Determination of the Section of MN.

The unsupported length is 204 inches, and the combined water and dead load stresses are 440,000 + 7,000 = 447,000 lb. compression. A column consisting of four 5" x 9/16" Z-bars and one 7" x ½" plate is assumed. The radius of gyration is 3.18 in, and $l/r = 63.5$. The allowable unit stress is 24,000 − 110 x 63.5 = 17,000 lb. per sq. in. The required sectional area is 447,000 ÷ 17,000 lb. = 26.3 sq. in. The assumed section gives an area of 27.26 sq. in., and it will be used.

Determination of the Section of EN

The unsupported length is 180 inches, and the water and dead loads are 340,000 + 800 = 340,800 lb. compression. A column is assumed which consists of four 5" x 3/8" Z-bars and a 7" x ½" plate. The radius of gyration of this section is 3.13, and $l/r = 58$. The allowable stress is 24,000 − 110 x 58 = 17,600 lb. per sq. in.

The area required is 340,800 ÷ 17,600 = 19.30 sq. in. The above section, giving an area of 19.90 sq.in., will be used.

DETERMINATION OF THE SECTION OF OF

The unsupported length of the member is 144 inches, and the combined stresses are 250,000 + 1,700 = 251,700 lb. compression. A column which consists of four 4" x 3/8" Z-bars latticed is assumed. The radius of gyration is 2.55 in., and l/r = 56.5 The allowable unit stress is 24,000 − 110 x 56.5 = 17,800 lb. per sq. in. The area required is 251,700 ÷ 17,800 = 14.10 sq. in. The assumed section, with an area of 14.64 sq. in., will be used.

DETERMINATION OF THE SECTION OF GP

The unsupported length is 120 inches, and the total stress is 260,000 + 1,300 = 261,300 ~~lb.~~ compression. The assumed column consists of four 4" x 3/8" Z-bars connected by latticing. The radius of gyration is 2.57 in., and l/r = 49. The allowable stress is 24,000 − 110 x 49 = 19,100 lb. per sq. in. The area required is 261,700 ÷ 19,100 = 13.70 sq. in. The assumed section, which gives an area of 14.64 sq. in., will be used for this member.

Determination of the Section of HR

This member is 96 inches long, and the dead and water load stresses are ,1000 + 264,000 = 26,5000 lb. compression. The section assumed consists of four 4" x 3/8" Z-bars. The radius of gyration is 2.57 in., and l/r = 37.5. The allowable stress is 24,000 − 110 × 37.5 = 19,870 lb. per sq. in. 26,5000 ÷ 19,870 = 13.40 sq. in., required area of the section. The assumed section, giving 14.64 sq. in., will be used.

The Sway Bracing

The eight 78-ft. bents will be braced in pairs as shown in Fig. 31. Each of the horizontal members consists of two $3\frac{1}{2}$" x $2\frac{1}{2}$" x 3/8" ∠s; and each of the diagonals consists of one 3" x 3" x 3/8" ∠. The braces in the plane of the bent, which are shown in Fig. 27, and the horizontal braces between the bents, are each made up of two $3\frac{1}{2}$" x $2\frac{1}{2}$" x 3/8" ∠s

General Drawings.

The general drawings on pages 67 and 68 show the general arrangement and details of the members in the bents and bracing and the footway, with the actual number of rivets.

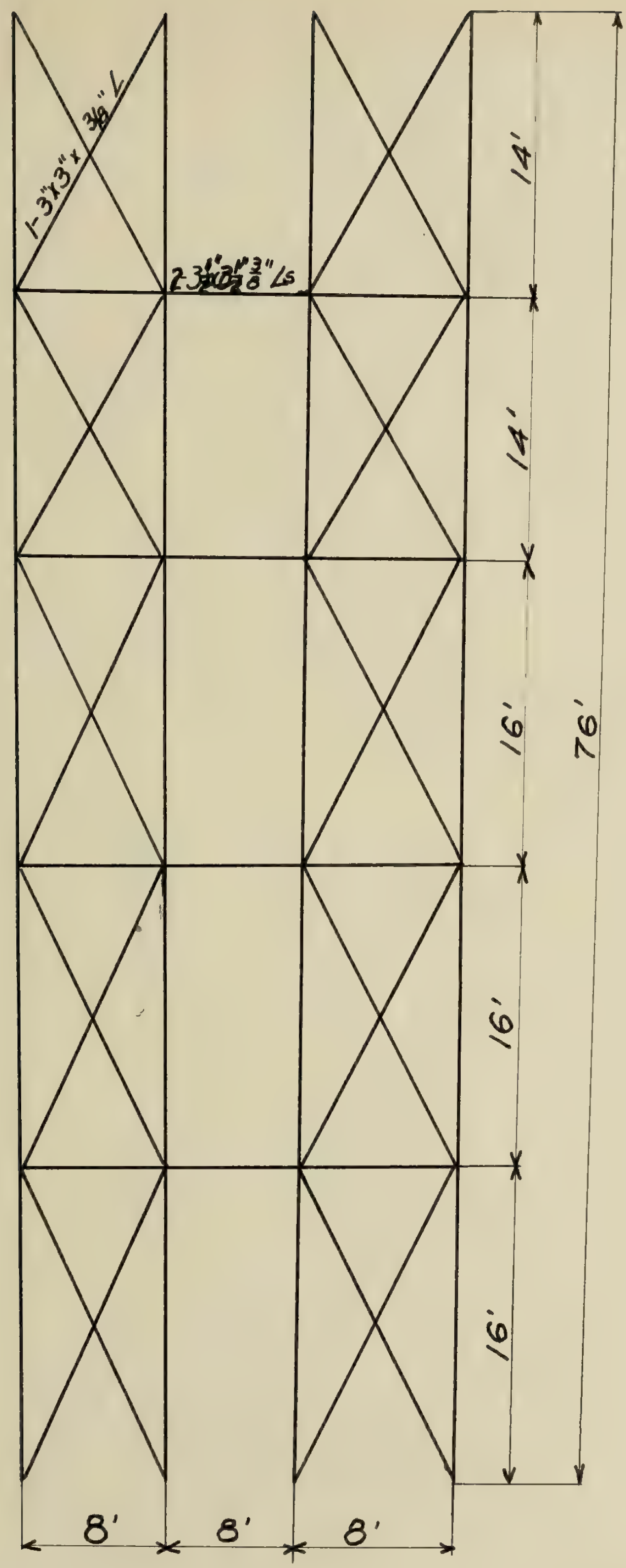
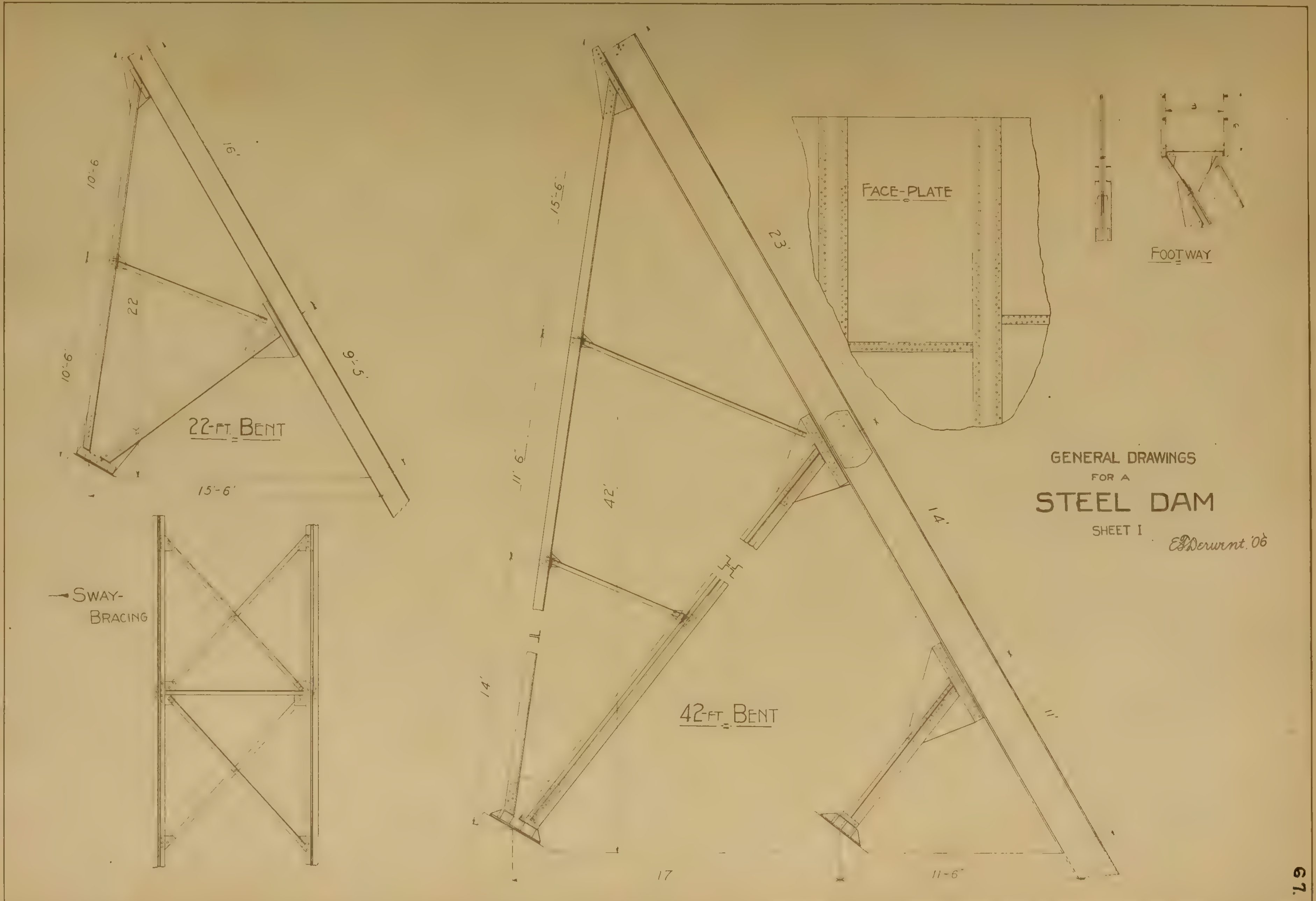
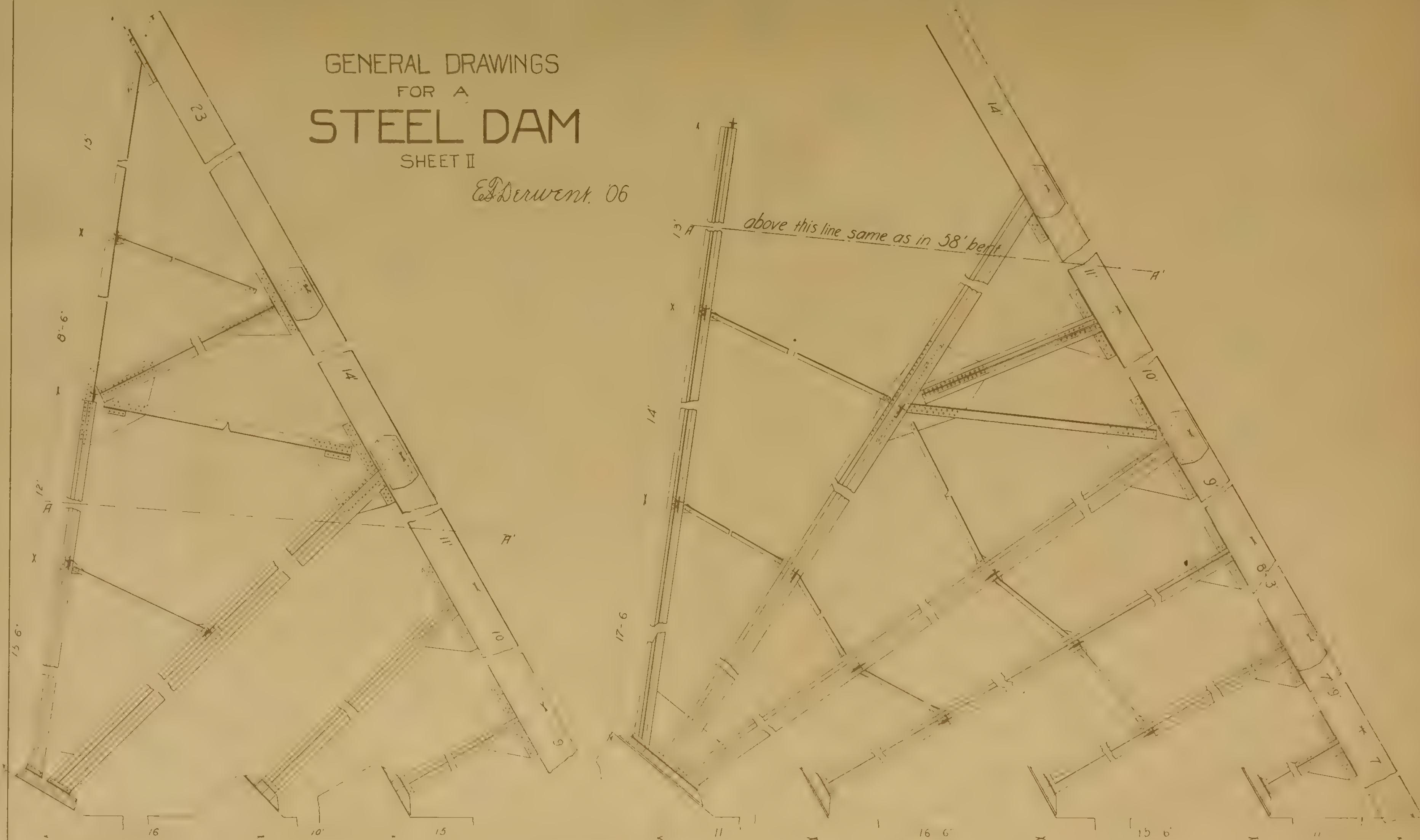


Fig. 31. Bracing for 78-ft. Bent.


GENERAL DRAWINGS
FOR A
STEEL DAM
SHEET I
E.F. Derwent. '06
FACE-PLATE
FOOTWAY
22-FT. BENT
16'
9'-5'
10'-6
10'-6
22
15'-6'
SWAY-
BRACING
42-FT. BENT
23'
14'
11'
15'-6'
11' 6'
42'
14'
17
11'-6"

GENERAL DRAWINGS
FOR A
STEEL DAM
SHEET II

E. D. Derwent. '06



58-FT. BENT

78-FT. BENT

THE ANCHORAGES.

The anchorage for the 22-ft bent has been described in connection with the design of that bent. For the other three bents, the I-beams on the water face are anchored to the masonry by eye-bars. Two bars are used for each bent, one on each side of the I-beam. They are connected to the beams by pins through the web, and are long enough to extend completely through the concrete, and three or four feet into the rock. They are laid parallel to the face of the wall, and at the lower end have eyes of the same size as those at the upper end. Through these lower eyes are passed bars 3 feet long, and of the same diameter as the pins in the upper ends. The webs of the I-beams are strengthened by pin plates.

The tension in the bottom panel of the beam in the 42-ft bent is 195,000 lb. The required sectional area is $195,000 \div 25,000 = 7.8$ sq. in. Two $4'' \times \frac{7}{8}''$ bars and a 4-inch pin will be used.

The greatest tension in the bottom panel

beam of the 58-ft bent is 250,000 lb. The area required in tension is 250,000 ÷ 25,000 = 10 sq. in. Two 5"x1" bars and a 5-inch pin will give a sufficient area and will be used.

The greatest tension in the bottom panel beam of the 78-ft bent is 360,000 lb. This requires 360,000 ÷ 25,000 = 14.4 sq. in. for tension. The use of two 7"x1" bars and a 6-inch pin is sufficient.

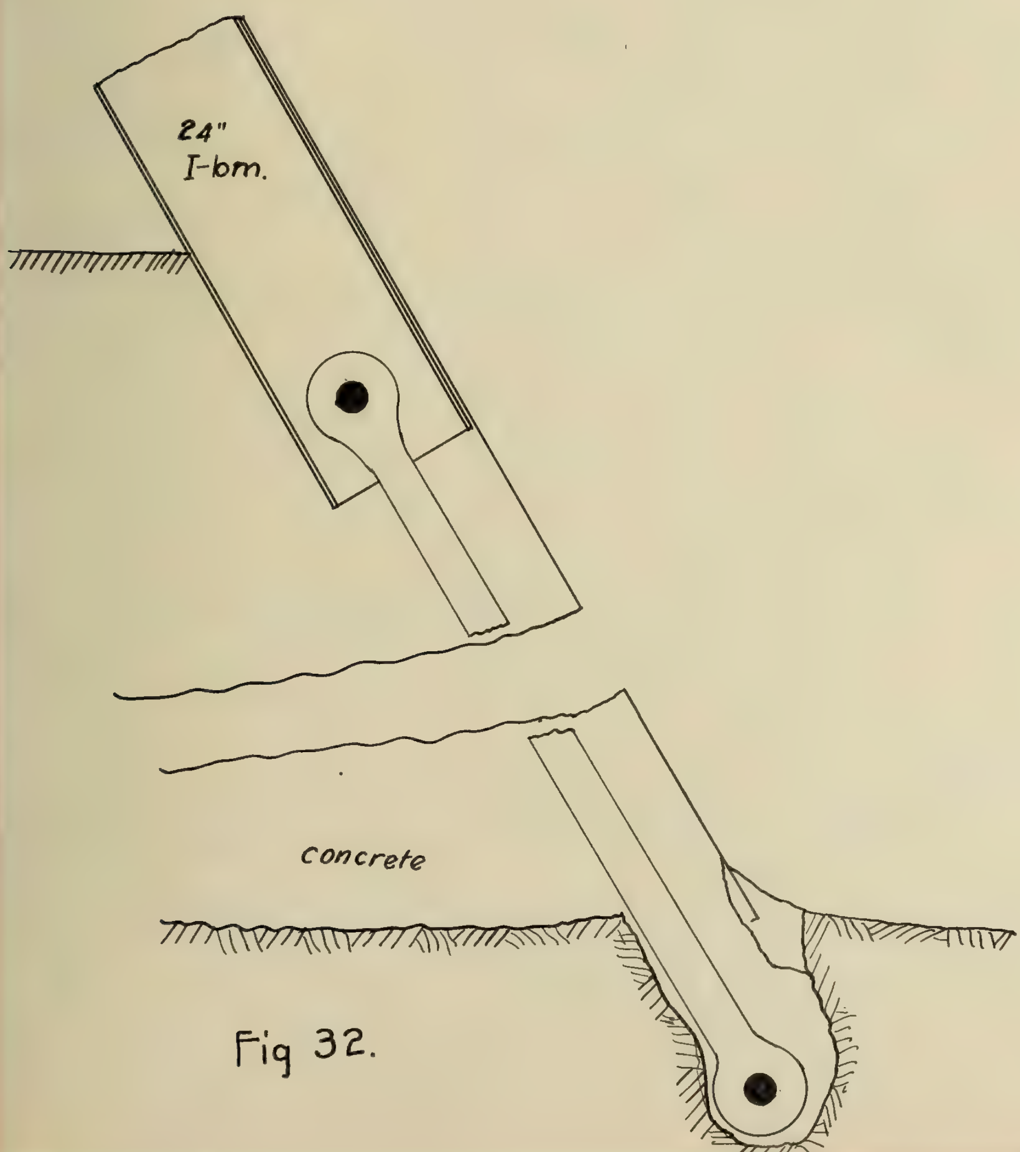


Fig 32.

Estimate of the Weight

Section	Length in ft.	Unit Weight in lb.	Total Weights.
The 22-ft Bent			
I-beam.	27.5	42.0	Wt of I-bm in 1 bent 1,155 8 Total wt. in 22-ft. 9,240 lb.
5" x 3½" x 3/8" ∠s	38.0	10.4	395.
6" x 4" x 3/8" ∠s	22.4	12.3	275.
6" x 6" x ½" ∠s	4.4	19.6	86.
3" x 3" x 3/8" ∠s	31.0	7.2	223.
3½ x 2½ x 3/8" ∠s	32.8	7.2	236.
3/8" x 36" pl.	6.7	45.9	305.
½" x 8" pl.	.5	13.6	7.
¾" x 27" pl.	1.2	68.9	86. pls. & ∠s = 1,613 8 Total, 12,904 lb.
1" rod	6.0	2.7	16 8 Total 128 lb.
The 42-ft. Bent			
24" I-beam.	58.0	100.0	5,800 10 For all 42-ft. bents 58,000 lb.
6" x 4" x 3/8" ∠s	76.6	12.3	942.
6" x 3½" x 3/8" ∠s	17.2	11.7	200.
3½" x 3" x 3/8" ∠s	5.6	7.9	44.
3½" x 2½" x 3/8" ∠s	89.0	7.2	641.

3"x 3"x $\frac{3}{8}$" Ls	50.0	7.2	362.	
$\frac{1}{2}$"x 50" pl.	4.5	85.0	383.	
$\frac{1}{2}$"x 48" pl.	3.7	81.6	300.	
$\frac{1}{2}$"x 31" pl.	2.9	52.7	153.	
$\frac{3}{8}$"x 18" pl	3.5	22.9	80.	pls + Ls = 3,115 10
				Total 31,150 lb.
4"x $\frac{3}{8}$" Z-brs.	224.0	12.4		2,780 10
				Total 27,800 lb.
1" rod	20.0	2.7		53. 10
				Total 530 lb.
cast iron pedestals, 2 @ 780# =				1,560 10 15,600 lb.

The 58-ft. Bents.

24" I-beam	74.0	100.0		7400 10
			For all 58-ft bents	74,000 lb.
6"x 4"x 3/8" Ls	50.8	12.3	625.	
6"x $3\frac{1}{2}$"x $\frac{3}{8}$" Ls	67.6	11.7	791.	
$3\frac{1}{2}$"x 3"x $\frac{3}{8}$" Ls	10.8	7.9	85.	
$3\frac{1}{2}$"x $2\frac{1}{2}$"x $\frac{3}{8}$" Ls	108.0	7.2	775.	
3"x 3"x $\frac{3}{8}$" Ls	66.6	7.2	480.	
$\frac{3}{8}$"x 6" pl.	22.0	7.65	168.	
$\frac{3}{8}$"x 7" pl.	15.0	8.93	134.	
$\frac{1}{2}$"x 7" pl.	27.0	11.90	320.	

$\frac{1}{2}$" x 20" pl.	4.0	34.00	136.	
$\frac{1}{2}$" x 44" pl.	5.0	78.80	394.	
$\frac{1}{2}$" x 42" pl.	4.0	71.40	286.	
$\frac{1}{2}$" x 32" pl.	5.5	54.40	300.	
$\frac{3}{8}$" x 32" pl	5.3	40.80	216.	
$\frac{3}{8}$" x 30" pl.	4.3	38.28	165.	
$\frac{1}{2}$" x 24" pl.	3.1	40.80	126.	
$\frac{3}{4}$" x 14" pl.	5.0	35.71	179	pls + Ls in 1 bent 5,180 / 10
				total in 58-ft bents 51,800 lb.
3" x $\frac{3}{8}$" Z-bars.	110.0	9.7	1070.	
4" x $\frac{3}{8}$" Z-brs.	172.0	12.4	2130.	
5" x $\frac{3}{8}$" Z-brs.	132.0	13.9	1830.	Z-brs in 1 bent 5,030 / 10
				total in 58-ft bents 50,300 lb.
1" rod	60.0	2.7		160 / 10
				Total, 1,600 lb
Cast iron pedestals 3 @	850	=		2,250 / 10
				Total, 22,500 lb

The 78-ft Bents.

24" I-beam.	96.0	100.0		9,600 / 8
				For all 78-ft bents. 76,800 lb.
6" x 4" x $\frac{3}{8}$" Ls	50.8	12.3	625.	
6" x $3\frac{1}{2}$" x $\frac{3}{8}$" Ls	117.2	11.7	1370.	
$3\frac{1}{2}$" x 3" x $\frac{3}{8}$" Ls	10.8	7.9	85.	

$3\frac{1}{2}" \times 2\frac{1}{2}" \times \frac{3}{8}"$ Ls	322.0	7.2	2320.		
$3" \times 3" \times \frac{3}{8}"$ Ls	91.1	7.2	652.		
$\frac{3}{8}" \times 6"$ pl.	45.0	7.65	344.		
$\frac{1}{2}" \times 7"$ "	65.0	11.90	770.		
$\frac{1}{2}" \times 20"$ "	4.0	34.00	136.		
$\frac{1}{2}" \times 44"$ "	5.0	78.80	394.		
$\frac{1}{2}" \times 32"$ "	5.5	54.40	300.		
$\frac{1}{2}" \times 30"$ "	16.0	51.00	815.		
$\frac{1}{2}" \times 36"$ "	6.5	61.20	400.		
$\frac{1}{2}" \times 48"$ "	7.0	81.60	570.		
$\frac{1}{2}" \times 40"$ "	6.0	68.00	410.		
$\frac{3}{4}" \times 14"$ "	8.0	35.71	285.	For 1 bent	9,477 8
				For all 78-ft bents	75,816 lb.
$3" \times \frac{3}{8}"$ Z-bars.	180.0	7.2	1280.		
$4" \times \frac{3}{8}"$ Z-bars.	364.0	12.4	4510.		
$5" \times \frac{3}{8}"$ Z-bars.	238.0	13.9	3300.		
$5" \times \frac{9}{16}"$ Z-bars.	124.0	20.2	2500.	For 1 bent	11,590 8
				For the 78-ft. bents.	92,720 lb.
1" rod	160.0	2.7			424 8
				Total,	3,392 lb
Cast iron pedestals.	3 @ 800 =		2400		
	1 @ 1500 =		1500		3,900 8
				Total.	31,200 lb.

The Facing.

6" x 1/2" Z-brs.	184.0	21.0	38,60.
5" x 5/16" Z-brs.	1800.0	11.6	21,000.

Total Z-brs. 24,860 lb.

1/4" pl. on concrete.	sq.ft. 5368.	10.2	54 700.
3/8" pl. on bents.	sq. ft. 19140	15.3	283 000.
1/2" x 18", splice pl.	2081	30.60	67 000.

Total plate 404,700 lb.

The Footway

3 1/2" x 2 1/2" x 3/8" ∠s	29.0	7.2	210.	
3/8" x 18" pl.	1.5	22.9	35. For 1 bent	245
				36
			For the dam.	8,800 lb.
1/2" pipe railing	1440	.84		1,210 lb.

The Anchorages.

1/2" x 2" bars	320.0	3.4	1090.
7/8" x 4" "	470.0	11.9	5600.
1" x 5" "	330.0	17.0	5600.
1" x 7" "	264.0	23.8	6300.
2" ⊙	20.0	10.68	214.
4" ⊙	30.0	42.73	1280.
5" ⊙	30.0	66.76	2000.
7" ⊙	24.0	96.14	2300.

For all bents 24,384.

ESTIMATE OF THE COST.

Raw. Material.

218,040 lb. I-beams @ $1.84½ =	$4,040.00	
195,680 lb. Z-bars. @ $1.89½ =	3,700.00	
585,170 lb. plates + angles @ $1.84½ =	10,800.00	
30,034 lb. bars. @ $1.75 =	525.00	
69,300 lb. castings @ 10¢ =	6,930.00	
1,440 ft. pipe @ 8¢ =	115.00	
29,967 lb. rivets @ 10¢ =	2,997.00	
Total.		$29,107.00

Mill Work.

218,040 lb. I-beams p. in web + fl. @ $.30 = 651.00

Drafting.

546 tons @ $1.00 = 546.00

Shop. Work.

780,850 lb. pls. angles and Z-bars @ $.75 = 5,860.00

Painting

521.7 tons @ 1 gal. for two coats per ton @ $1.00 = 522.00

Freight and Erection

564.7 tons @ $10.00 = 5,647.00

Net cost	$42,333.00
Profit, + 15%	6,348.00
Total cost of metal	$48,681.00

The pound price for the metal is,

$$48,681^{00} \div 1,219,401 = 4.3 \text{ cents per lb.}$$

<u>The Concrete</u>.

4200 cu. yds @ $\$6^{00}$ per yd = $\$25,200^{00}$

The total cost of the dam is

$$48,681^{00} + 25,200^{00} = \$72,081^{00}$$

CONCLUSION

The cost of the masonry dam which ~~is~~ now in place was $\$88,400^{00}$, making a difference in favor of the steel dam of $\$16,319^{00}$. This shows the steel dam to be 18.4 % cheaper than the masonry dam. The steel is thus shown to be a cheaper material than masonry even in a locality where stone was easily obtained and freight on steel was high. In addition to the item of cheapness in its favor, the steel dam could be built in much less time and the expenses for engineering and superintendence should be less than for the masonry dam. With proper care, the steel dam should last as long as the masonry dam. It would be always impervious and in general should give good satisfaction.